Manual for Railroad Engineers

Geo. L. Vose.

Reconnoissance.
Survey.
Location.
Laying out Work.
Earthwork.
Rockwork and Tunnelling.
Strength of Materials.
Strains in Girders.
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MANUAL

FOR

RAILROAD ENGINEERS

AND

ENGINEERING STUDENTS.

CONTAINING

THE RULES AND TABLES NEEDED FOR THE LOCATION, CONSTRUCTION, AND EQUIPMENT OF RAILROADS, AS BUILT IN THE UNITED STATES.

BY

GEORGE L. VOSE,

PROFESSOR OF CIVIL ENGINEERING IN BOWDOIN COLLEGE.

WITH ONE HUNDRED AND SIXTY-FIVE WOOD CUTS, AND THIRTY-ONE LARGE PLATES.

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PREFACE.

THE following pages are prepared both for the use of railway engineers and engineering students. For the first class, the aim has been to give reliable information upon the various matters relating to the location, construction, and equipment of railroads, in such a manner that the actual numerical results required in every-day work may be at once obtained. Scientific discussions of engineering problems have purposely been avoided, as such are given at length in the works of Rankine and others. Very little of mathematics has been introduced, as many members of the profession will at once close a book which requires the study of complex formulæ. Even accomplished mathematicians have little time to stop in the midst of engineering practice to deduce a required rule from first principles. It is believed that there is nothing in this work that may not be easily comprehended by any intelligent roadmaster. Frequent references are made to the standard works upon engineering, in order that those who wish to follow the rules back to their source may do so. The greater part of the matter has been drawn from the actual practice of American engineers, and the numerous illustrations have beer selected from works of tried and approved merit. Quite a large part of the work is devoted to those technical details, upon the correct execution of which so much of the success of engi-

neering practice depends The actual dimensions figured upon the large plates, being the result of the experience of our best engineers, will be readily appreciated.

For the engineering student, the aim has been to present, in a connected manner, the various operations that go to make up the laying out and building of a railway, and at the same time to give him an idea of those modifications which his abstract science must receive before it is fit to be applied to practice. There are many problems in practical engineering embracing so large a number of variable elements that it is simply impossible for science to solve them in a satisfactory manner. In such cases we may often, by accumulating a large amount of experience, deduce an empirical rule, which will answer every requirement; and it is important that the student should be able both to accumulate and arrange the results of practice, and to draw therefrom rules for his own guidance.

Some points in Railway Engineering are but lightly touched upon in the following pages. The methods of laying out railway curves, the numerous problems in track laying, and the important matter of computing the quantities of excavation and embankment, have been so fully and so well treated by Henck, Trautwine, and Morris, that there is no reason for doing the same work over again. At the risk of appearing merely to return a compliment, the author would take this opportunity of referring to the later work of Mr. Trautwine, viz., The Civil Engineer's Pocket Book, as being beyond all question the best practical manual for the engineer that has ever appeared.

For the benefit of students, the following list of works bearing upon the subject of Railway Engineering is given. It contains only works easily procured, not very expensive, and of real value,

which may be profitably used by those possessing but a limited amount of mathematical knowledge.

1. Field Book for Railroad Engineers. By John B. Henck.
2. The Plane Table, and its Use in Topographical Surveying. From the papers of the United States Coast Survey.
3. The Principles and Practice of Levelling, applied to Railway Engineering, and to the Construction of Roads. By F. W. Simms.
4. A New Method of Calculating the Cubic Contents of Excavation and Embankment, by the Aid of Diagrams. By John C. Trautwine.
5. A Manual of the Principles and Practice of Roadmaking. By William M. Gillespie.
6. An Elementary Course of Civil Engineering. By D. H. Mahan.
7. The Mode of Estimating the Stresses in Bridges and Roofs, by Means of Diagrams. By Robert H. Bow.
8. The Strains on Structures of Iron-work. By F. W. Shields.
9. An Elementary and Practical Treatise on Bridge-building. By S. Whipple.
10. The Theory of Strains in Girders and Similar Structures; with Observations on the Application of Theory to Practice, and Tables of the Strength and other Properties of Materials. By Bindon B. Stoney.
11. Graphical Method for the Analysis of Bridge Trusses, extended to Continuous Girders and Draw Spans. By Chas. E. Greene.
12. Elementary Principles of Carpentry. By Thomas Tredgold. A new edition, edited by John Thomas Hurst.
13. A Practical Treatise on Limes, Hydraulic Cements, and Mortars. By Q. A. Gillmore.
14. The Kansas City Bridge. By O. Chanute and George Morrison.
15. An Account of the Iron Railway Bridge across the Mississippi River, at Quincy, Illinois. By Thomas Curtis Clarke.
16. Locomotive Engineering and the Mechanism of Railways. By Zerah Colburn.

For periodical literature, the reader is referred to the two English publications, "The Engineer," and "Engineering;" and to

Van Nostrand's Eclectic Engineering Magazine, and The Journal of The Franklin Institute, in this country Works like Brees' Railway Practice ($75.00), Humber's Treatise on Cast and Wrought Iron Bridge Construction ($60.00), and Perdonnet et Polonceau, Noveau Porte-Feuille de l'Ingenieur des Chemins de Fer ($90.00), both on account of their cost and their size, are more suitable for the libraries of public institutions than for those of students.

The author is especially indebted to the following gentlemen for materials employed in the preparation of this work: Samuel S. Montague, Chief Engineer of the Central Pacific Railroad, T. E. Sickles, General Superintendent of the Union Pacific Railroad, Messrs. Chanute and Morrison, of the Kansas City Bridge, Messrs. Fink and Vaughan, of the Louisville Bridge, Willard Pope, of the Detroit Bridge and Iron Works, B. H. Latrobe, Esq., Messrs. Smith and Latrobe, and Wendel Bollman, of Baltimore, Edward H. Williams, formerly General Superintendent of the Pennsylvania Railroad, at present of the Baldwin Locomotive Works, T. C. Clarke, of the Phœnixville Bridge Works, J. H. Linville, of the Keystone Bridge Works, Messrs. J. M. Wilson and H. Pettit, of the Engineer Department of the Pennsylvania Railroad, Charles Hilton, Engineer of the Hudson River Bridge Company, Benjamin D. Frost, State Engineer at the Hoosac Tunnel, George A. Parker, Esq., formerly Chief Engineer of the Philadelphia, Wilmington and Baltimore Railroad, Edward S. Philbrick, formerly Chief Engineer of the Boston and Worcester Railroad, John F. Anderson, Chief Engineer of the Portland and Ogdensburg Railroad, George F. Morse, Superintendent of the Portland Locomotive Works, and to John A. Haven, Publisher of the American Railway Times.

The plates accompanying this work have been engraved upon

wood, by John Andrew and Son, of Boston, and very much of their value depends upon the faithful manner in which they have been executed. The engraver's task has been by no means an easy one, as many of the plates have been reduced by photography from large working drawings, and from tracings containing a very considerable amount of detail.

GEORGE L. VOSE.

BRUNSWICK, ME., August, 1872.

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MANUAL

FOR

RAILROAD ENGINEERS.

INTRODUCTION.

Rise and Progress of the Railway System.

No precise time can be assigned for the birth of the railroad system. From the year 1630, when the first rude tramways were made, to the memorable competition at Rainhill, in 1829, when the success of the "Rocket" proclaimed to the world the advent of a new and mighty agent of civilization, many minds were actively engaged upon the solution of the great problem of rapid and cheap transport for the continually increasing quantities of material used in the various arts and manufactures. The most marked advance, however, was made between 1820 and 1830 — a period that witnessed the building of the roads from Stockton to Darlington, and from Liverpool to Manchester; that beheld the commencement of our own vast network of railways; that saw the system inaugurated which at this day is represented by an amount of road sufficient to reach six times around the world, and which has cost not less than fourteen thousand millions of dollars.

Passing over the early days of *tramways*, the railroad system may be said to have been fairly before the public in the fall of

1825. At that time a locomotive steam engine upon the Stockton and Darlington road, built and run by George Stephenson, drew a train of wagons conveying four hundred and fifty passengers, besides freight, a distance of eight and three quarters miles in sixty-five minutes. The engine used upon this occasion was, however, an exceedingly imperfect machine. It was not until four years later, in October, 1829, that the "Rocket" gained the prize offered by the Directors of the Liverpool and Manchester Railway, and forever settled the question as to the superiority of the locomotive steam engine as the motive power. In America, the period from 1825 to 1832 saw the construction of the Quincy Granite Railway in Massachusetts, the Mauch Chunk coal road in Pennsylvania, and the Mohawk and Hudson Railway in New York. The same period beheld the commencement of the Baltimore and Ohio Railroad, of the line from Charleston, in South Carolina, to Augusta, in Georgia, and of the several roads from Boston to Lowell, Providence, and Worcester. The early days of the new mode of transport in this country were not marked by the severe struggle which preceded its birth in England. The grave discussions, however, as to the employment of horses upon railways, the doubts as to the possibility of being able to surmount a grade of thirty feet in a mile, and of running trains in bad weather, and, finally, the estimates of the amount of traffic which might be expected, all serve to show how little the projectors of our iron roads comprehended the capacity and power of the system which in less than fifty years should connect the Atlantic with the Pacific; which should open the granaries of the West to the crowded cities of the Old World; which should almost entirely supersede, both for passengers and freight, every other mode of inland transportation.

The rate of progress in railroad construction, in the United States, from 1830 to 1874, has been as follows:—

Year.	Miles open.	Year.	Miles open.
1830 . . .	23	1855 . .	18,375
1835 . . .	1,098	1860 . .	30,635
1840 . . .	2,818	1865 . .	35,185
1845 . . .	4,633	1870 . .	53,000
1850 . . .	9,021	1874 . .	70,000

The table below gives the rate of progress, in the different sections of the country, from 1840 to 1870.

Year.	New England States.	Middle States.	Western States.	Southern States.
1840 . .	527	1,566	89	636
1845 . .	973	2,100	374	1,186
1850 . .	2,508	3,202	1,276	2,035
1855 . .	3,469	5,473	4,567	4,857
1860 . .	3,660	6,706	11,064	9,182
1865 . .	3,834	8,539	12,847	9,632
1870 . .	4,494	10,991	23,769	12,468

Cost of Railways.

The roads of New England have cost, upon an average, $50,000 per mile; those of the Middle States, $75,000; of the Western, $50,000; and of the Southern, $35,000. The general average for the whole country is about $50,000 per mile; making a total, in round numbers, of $3,500,000,000 for the whole 70,000 miles of railway. These figures, however, are to be regarded as only approximate; being based upon the capital accounts, and not upon the amounts actually expended. The stocks and bonds of the several companies have seldom realized their nominal value. Thus the cost per mile, as returned, does not show the actual relative expense of construction in different sections of the country, on account of the wide difference in the percentages realized from the sale of the securities of the various companies in different localities. The New England roads appear, from the above figures, to have cost no more per mile than the roads of the

Western States; while, regarding the amount actually expended for construction, the Western roads are much cheaper than those of New England.

THE SERVICE PERFORMED BY RAILWAYS.

The amount of work performed in a year by the whole railway system of this country can be estimated only roughly, since but few states make annual returns of the operations of their roads. If our whole 70,000 miles averaged only half as much traffic, and earned only half as much per mile, as the roads of New York State did in 1868, the whole number of passengers carried in a year would be 160,000,000; the number of tons of freight 108,000,000; the total miles run by trains 225,000,000; and the revenue $450,000,000. The passenger mileage* would have been 6,000,000,000; and the freight mileage 12,000,000,000. To put these last figures into more popular language, if all of the railway travelling done in the United States in a year were done by one person, the actual distance accomplished would carry that person from the earth to the sun and back again thirty-two times; or it would carry him 240,000 times around the world, and he would travel at the rate of 12,000 miles a minute night and day. The total freight mileage performed in the United States for a single year, as above given, would be equivalent to transporting the great pyramid of Egypt from New York to St. Louis and back again. If the locomotives and cars employed in doing all of this work were made up into a single train, it would reach not less than two thousand miles. The actual value of the freight transported in a single year in the United States has been reckoned at upwards of $10,500,000,000; or about three times the cost of all the roads in the country.

* By mileage is meant the product of miles run by passengers, or by tons carried. Thus, 500 persons carried 100 miles, and 750 persons carried 75 miles gives a total passenger mileage of—

$$500 \times 100 + 750 \times 75 = 106,250.$$

The following table shows the extent and the approximate cost of the railways of different countries at the close of 1873:—

	Length in Miles.	Cost per Mile in Dollars.	Total Cost in Dollars.
United States, . .	70,000	50,000	3,500,000,000
Great Britain, . . .	16,000	180,000	2,880,000,000
France, ...	11,000	160,000	1,760,000,000
Germany, ...	13,000	125,000	1,625,000,000
Austria, ...	7,500	80,000	600,000,000
Russia, ...	8,500	160,000	1,360,000,000
Italy, ...	4,000	100,000	400,000,000
Spain and Portugal, .	4,000	100,000	400,000,000
The rest of Europe, .	4,000	90,000	360,000,000
India, ...	5,000	95,000	475,000,000
Africa, ...	1,000	125,000	125,000,000
Australia, ...	1,000	105,000	105,000,000
South America, . .	2,000	100,000	200,000,000
Canada, ...	3,000	70,000	210,000,000

The whole length of the railways of the world was thus 150,000 miles; and the total cost, in round numbers, \$14,000,000,000. This vast system of conveyance is reckoned to transport annually 1,000,000,000 passengers, and 500,000,000 tons of freight, and to give regular employment to over a million persons.

The Effect of Railways.

The effect of a judicious system of railways is to increase the consumption and to stimulate the production of agricultural products, to distribute more generally the population, to cause a more even balance between supply and demand, and to increase largely both the amount and safety of travelling. By railroads, large cities are supplied with fresh produce from the country, and persons are enabled to live at a distance from the great

centres, and yet do business therein. The amount of this diffusion of the population is as the square of the speed of transport. If a person walks three miles an hour, and has an hour for passing from his residence to his place of business, he cannot reside at a greater distance than three miles from his work. The area, therefore, in which such persons may live, is the circle of which the radius is 3 miles, the diameter 6 miles, and the area 28 square miles, very nearly. If, by the use of a horse, the speed is increased to 8 miles an hour, the diameter of the circle becomes 16 miles, and the area 201 square miles. Finally, if by railway the speed is 25 miles an hour, the diameter becomes 50 miles, and the area 1,963 square miles.

The most marked benefit derived from an improved mode of transport is seen in the increased value which is at once put upon the products raised at a distance from the great markets. Suppose a ton of wheat to be worth $50, and a ton of corn $25, at market. Suppose also that it costs 20 cents per ton per mile for transport over a common road, and 2 cents per ton per mile for transport upon a railway; then the values of wheat and corn, raised at different distances from market, will be as shown below.

	Wheat.		Corn.	
	Transported by Railway.	Transported by Wagon.	Transported by Railway.	Transported by Wagon.
At market,	$50	$50	$25	$25
50 miles,	49	40	24	15
100 miles,	48	30	23	5
150 miles,	47	20	22	0
200 miles,	46	10	21	0
250 miles,	45	0	20	0
300 miles,	44	0	19	0

Thus the transport of a ton of wheat, carried 250 miles upon a common road, would just equal its value; or, in other words, at 250 miles from market wheat would have no value for export; while carried the same distance by railroad, the transport con-

sumes only $5 per ton, thus giving it a value of $45 at a distance of 250 miles from market. So too with regard to corn: if it has a value of $25 per ton at market, at 125 miles distant upon a common road it is worth nothing at all, as the cost of taking it to market just consumes its value.

The radius of the circle in which corn may be raised with profit, if it is obliged to rely upon wagon transport, is then 125 miles, and the area is 49,087 square miles; while the radius of the circle which may be profitably cultivated for railroad transport is 1,250 miles, and the area 4,908,750 square miles. In the same way the area for wheat, by wagon transport, is 196,350 square miles, and the area by railway transport, 19,635,000 square miles.

The Safety of Railway Travelling.

Notwithstanding the occasional disasters upon railways, there is no other mode by which travellers can be carried so safely. To be convinced of this we have only to consider the number of lives lost as compared with the whole number of passengers carried. Carefully collected returns show that the number of persons killed and injured, from causes beyond their own control, has been, in France, one in every 4,000,000 carried; in Prussia, one in 3,000,000; in Belgium, one in 1,600,000; and in Massachusetts one in 1,475,000. In 1866, the whole number of persons carried on the railways of Great Britain was over 300,000,000; while only fifteen were killed, or one in every 20,000,000. The Massachusetts returns show one passenger killed for each 25,000,000 carried, and one person injured for each 1,500,000 carried; while one and a half million persons have been transported for every one killed or injured from causes beyond the passenger's control. The government returns in France show the number of persons killed and injured while travelling by stage-coaches to be one in 28,000 — making it thus about one hundred times safer to travel by railway than by stage-coach. The danger of travelling by steamboat, as far as we can judge by the very incomplete returns, is far greater even than by stage.

Determination of the Character of the Road.

In commencing a railway enterprise, we should ascertain, in the first place, all that can be known of the amount and kind of traffic that is to be expected: whether passenger, freight, or mixed; whether the road is to depend principally upon a local business, or is to form a portion of a great trunk line; whether the traffic is to pass mostly in one direction, leaving the empty cars to be hauled back, or whether an equal amount of work may be expected in both directions; for upon the nature and extent of the traffic must depend the character of the road to be built. Upon a correct idea of what the road ought to be, depends in a great degree its success. The greater the amount expended in reducing the natural surface, to obtain low grades and easy curves, the less will be the cost of operating the road, but the greater the interest to be paid upon the first outlay. The limit of expenditure must be such as to render the sum of the construction and of the maintaining capitals a minimum, *eventually*; though it may not be expedient to produce this result at the outset. A road may be made for $50,000 a mile, and cost $9,000 per mile per annum for working and keeping in repair; or it may be made for $100,000 a mile, and cost only $6,000 per mile per annum for maintenance and operation. The interest on the cost of building added to the annual expense for operation will in each case be the same; and yet it may be preferable to adopt the first, for various reasons. It is easy, if the amount of traffic warrants it, to improve a cheaply made road to any desirable extent after it is opened; but if the traffic should turn out small, it would not be easy to pay the interest upon the cost of an expensive railway. Nothing would have been more absurd than for American engineers to have copied the enormously extravagant method of railway building adopted in England. Had it been done, we never should have had the immense system that has so developed the vast resources of our interior. Yet nothing could be wiser than for the managers of American roads to bring their lines gradually into that

admirable condition of efficiency which enables the European railways to be maintained for less than half the expense of ours. A cheaply built road may often be made into a first-class railway by a judicious expenditure of its earnings, at much less expense than would be incurred by a large outlay at first, when, as is generally the case, money is raised at a large sacrifice of the company's securities.

The determination of the increase of traffic which may be expected in any district from the building of a railway is a difficult matter. There can be few rules given for proceeding in such an inquiry. Experience is the only guide ; and even the experience in one district must be cautiously used in judging of another. There are many roads in this country which have been of incalculable benefit to the sections through which they pass, while the original stock is worth nothing, and never will be. Probably there is not a state in the Union that could not better afford to assume the cost of its railways than to be without them.

In fixing the general character of the road, as to gauge, grades, and curves, many points will have to be considered. The superiority which the modern railroad possesses over the common road, consists, first, in the reduction of the resistance to motion, and, second, in the application of the locomotive steam engine. The effect of a grade of a given inclination is relatively more upon a railroad than upon a common road, for as the absolute resistance upon a level decreases, the relative resistance of a given grade increases ; whence to obtain the full benefit of the railroad system, we must reduce the grades much more than upon a common road. For example, if the resistance to moving one ton upon a level on a railway is 10 pounds, and upon a common road 60 pounds, where a 24 feet grade, or an incline rising 24 feet in a mile, would double the resistance upon the railway, a grade of 141 feet per mile would be required to do the same upon the common road. The bad effect of grades, however, is by no means so great as at one time supposed, as only a part of the working expense is affected thereby. With regard again to curves, the amount and

sharpness of curvature will depend much upon the character of the trains which are to be drawn over the road. Long freight trains or fast passenger trains demand a large radius of curvature; while light traffic and moderate speeds are but little affected by curves of 2000 feet radius, and upwards. The various improvements introduced into the construction of locomotives and of cars during the past twelve years have very much reduced the cost of working both grades and curves. This matter will, however, be discussed in advance.

The Gauge of Railways.

The question as to the proper gauge * or width of track, which was formerly a favorite subject for dispute among engineers, has finally settled itself in favor of the narrow system, i e., 4 feet $8\frac{1}{2}$ inches: not that this particular width is absolutely the best, but that it is the one most commonly adopted, and that the advantages of uniformity so far outbalance all other considerations. Probably a gauge of 5 feet would have been found as nearly perfect for the railway system, as a whole, in a mechanical point of view, as any other; and it is much to be regretted that it was not fixed as a national standard in the commencement. The 7 feet gauge of the Great Western Railway of England has proved itself a failure, and is being rapidly reduced to 4 feet $8\frac{1}{2}$ inches. So, too, in America, the 6 feet of the Erie and of the Ohio and Mississippi has shown the fallacy of the reasoning which led to its adoption, and these roads will eventually, without doubt, be reduced to the narrow gauge.† With regard to the gauges of $5\frac{1}{2}$ feet, 5 feet, and 4 feet 10 inches, in the United States, every year is showing in the plainest manner the results, to one of which the railroads thus built must come; either a reduction to the 4 feet $8\frac{1}{2}$ inches, or else complete isolation from the general system. The

* The gauge is the width between the insides of the heads of the rails.

† Since writing the above, the gauge of the last named road has been reduced to the ordinary width of 4 feet $8\frac{1}{2}$ inches.

heaviest traffic in the world, and the most powerful engines, are to be found upon the narrow gauge. A commission of the best engineers of England have stated that engines can be run with perfect safety at any speed from 30 to 60 miles an hour on a gauge of 4 feet 8½ inches. Whether the total resistance to traction is any less upon the broad than upon the narrow track is yet to be determined. Even if it is slightly less, the widening of the road and all its equipment is rather an expensive mode of obtaining such an advantage. One of our prominent railroad managers, having operated both the 6 feet and the narrow gauge, concludes that in the cost and efficiency of the equipment there is a difference of 20 per cent in favor of the latter.* Indeed, at the present time many engineers favor a gauge of only three feet for certain roads; and even a less width than this has been adopted in some parts of Europe, with a good degree of economy in operation.†

General Establishment of the Railway Route.

The straight line connecting any two places would of course be the best for the completed road. But this is seldom practicable. Way towns must be accommodated to a certain extent. It should be borne in mind, however, that in adding to the length of a road we not only increase the cost of construction and of maintenance,

* "The advocates of the broad-gauge theory of late years have been pretty much confined to its unfortunate victims. The extra weight of its rolling stock, in proportion to the load carried, raises the proportion of non-paying tonnage; and the friction due to the curves is largely in excess of that upon the narrow gauge. In some foreign countries, where such property is protected from competition, the broad gauge roads are prosperous; but where they come in open competition with the narrow gauge, bankruptcy is the rule, to which there is hardly an exception. Of the four thousand miles of broad gauge roads in this country, I know of but one line that is not hopelessly bankrupt, and that makes two or three per cent. by carrying its through freight on a narrow gauge, for which it has laid down a third rail. Without the third rail, it would be as bankrupt as the rest of them." — J. W. Brooks: *Testimony before Committees on Railroads of Mass. Legislature*, 1870.

† See Appendix. "Narrow Gauge Railways."

but that we must move the entire through traffic over such increased length. In judging of the propriety of deflecting a road to pass through any town, we should be guided not by the present but the prospective importance of such a place. Almost any town possessing manufacturing or commercial advantages will pay a valuable tribute to the railway, provided the latter places itself in a position to stimulate the town, and to receive the tribute. So direct and sure is the action and reaction between the railway and the district through which it passes, that we may truly say, in the language of one of the earliest friends of the system, "Let the country but make the railway, and the railway will make the country."

CHAPTER I.

RECONNOISSANCE.

THE object of the Reconnoissance is to determine approximately the place for the road, to find the general form of the country, and to select that route which, with reference to the expected traffic, shall give the best arrangement of grades and curves; to determine the height of the controlling points in the various routes examined, and, in fine, to prepare the way for the survey in detail.

Routes placed upon the immediate bank of a large stream, will intersect a great number of lateral tributaries, which must be crossed by the road.

Routes placed upon the slopes of hills, are more subject to sliding and washing of the earthwork than when placed upon plateaus or in bottom lands.

Routes crossing the dividing ridges between the principal watercourses, ascend and descend to a greater or less extent, thus rendering necessary a strong motive power to work the road.

The general topography of a country may be ascertained by reference to the common maps, when such exist, or by riding over the district to be examined. The direction and size of the watercourses will show at once the position of summits.

Water flowing as in Fig. 1 indicates a fall from C to D, and also transverse slopes from A A A, and from B B to the bed of the stream. Fig. 2 shows a broken ridge, A A A A, from which the water flows in both directions towards C and D.

If it be required to join the points A and D (Fig. 3), we may pass at once through B and C, or we may go by the streams

L M and O P. By the latter route the road would ascend all of the way from A through L and M to the summit at N, and descend all the way from N through O and P to D. If the

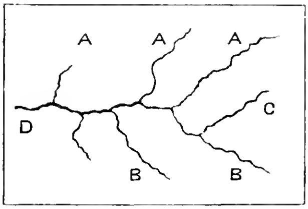


Fig. 1.

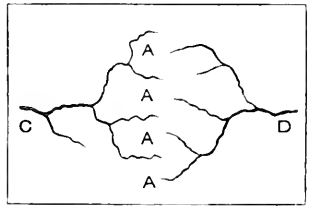


Fig. 2.

country between B and C is an elevated plateau, the profiles or sections of the two routes would be as shown in the sections A B C D and A N D.

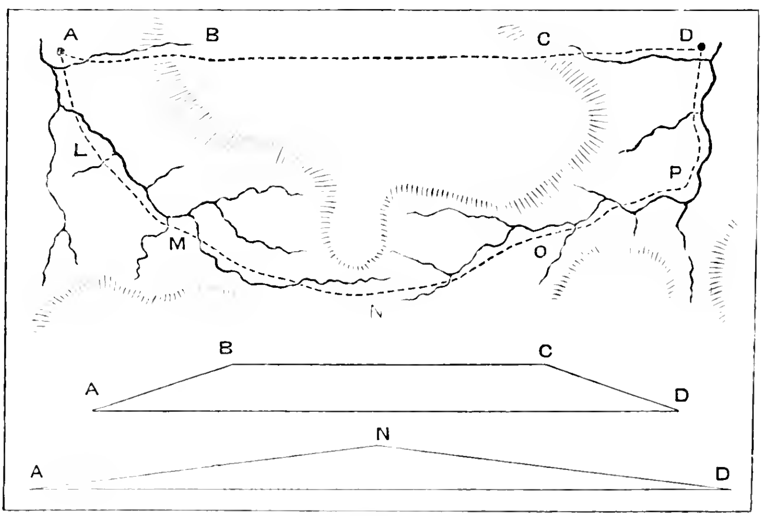


Fig. 3.

In passing from A to B (Fig. 4), by the several routes A B C D, we should have the corresponding profiles shown in Fig. 5;

from which it appears that the nearer to the head of the streams we cross, the less is the difference of elevation to be overcome.

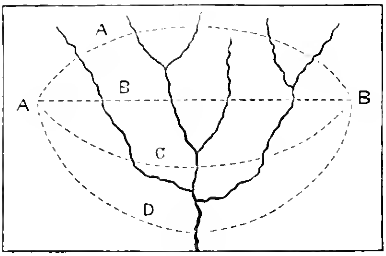


Fig. 4.

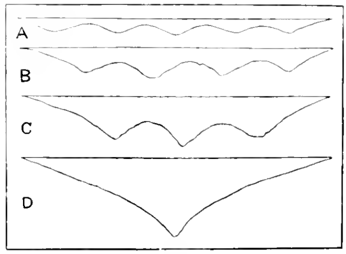


Fig. 5.

Barometrical Measurement of Heights.

The relative height of summits, the rate of fall of streams, and absolute elevations within a few feet, may be easily and rapidly ascertained by means of the barometer. The principle involved in this mode of levelling is, that the density of the air, by which the earth is surrounded, decreases as we ascend in a fixed and known ratio to the elevation; and the practice consists in measuring precisely this density. The most portable and convenient barometer for engineering purposes is that known as the Aneroid, or Holosteric.* This instrument consists of a flat cylindrical metallic box, exhausted of air, the top of which is made very thin, and corrugated in concentric circles, in order to render it more elastic When the atmospheric pres-

* For a description of the Standard, Mercurial, Cistern Barometer, and the method of using it for the measurement of elevations, with examples from actual practice, the reader is referred to Professor Guyot's tables in the Smithsonian Miscellaneous Collections; to the Manual of the Mercurial and Aneroid Barometers, by J. H. Belville, of the Greenwich Observatory; and to an excellent pamphlet upon the Barometer, Thermometer, &c., by Commodore Thornton A. Jenkins, Chief of the Bureau of Navigation, U. S. Navy Department.

sure increases, this corrugated top is forced inwards or downwards. When, on the other hand, the atmospheric pressure decreases, the elasticity of the metal, aided by a spring or by a counterweight, tends to move it in the opposite direction. This movement of the top of the box is conveyed by a series of multiplying levers to an index moving over a circular scale graduated to correspond with the standard barometer. The several parts of this instrument are shown in Fig. 6. A A is the metallic box,

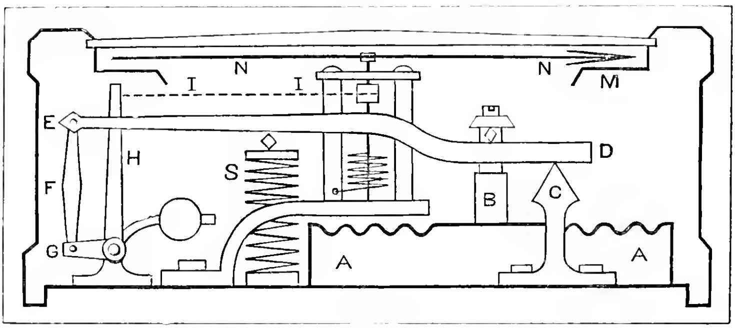


Fig. 6.

with corrugated top, exhausted of air, and fixed to the bottom of a brass case enclosing the mechanism of the whole instrument. B is a small column, connecting the top of the box with the principal lever E D, the latter moving upon the fulcrum C. The movement of the small end of the lever is carried by the rod F to the short end of the bent lever G H, to the upper end of which is attached the watch chain I I. This chain passes round a small drum upon the arbor carrying the needle N N at its upper end. A small hair spring upon the arbor regulates the motion of the needle. S is a spiral spring, which, by its tension, raises the principal lever D E, when the pressure upon the top of the box is in any way lessened. The circular scale, seen in section at M, is graduated by comparing its indications under different pressures with those of a standard mercurial barometer. With a good Aneroid a difference of ten feet in elevation may be

detected. The mercurial column in the cistern barometer falls, in round numbers, one inch for each 1000 feet of ascent. The amount of motion of the Aneroid needle corresponding to one inch of the mercurial column depends upon the size and proportions of the instrument.* With the outer case five inches in diameter, the needle is three inches long, and the diameter of the graduated circle the same. One inch on the mercurial column is represented upon such a circle by an inch and a half. This inch and a half is called an inch, and is divided into ten parts; and each of these again is subdivided into five parts; and as these smaller divisions are easily halved by the eye, the one hundredth of an inch, which corresponds to a difference in elevation of only ten feet, is readily determined.

To use the Aneroid, the following rule has been prepared:—

As the *sum* of the readings, at the different stations, is to their *difference*, so is 55,000 to the elevation required. Thus, if the reading at the foot of a hill is 30.05, and at the top 29.44, the *sum* is 59.49, and the *difference* 0.61; whence the proportion, 59.49 to 0.61, as 55,000 to 564 feet.

At the back of the Aneroid is placed a small screw, by means of which the needle may be *set* in either direction, so as to correspond with a standard barometer. In measuring an elevation, or in running lines of levels, the Aneroid should be compared with a standard barometer, or with another Aneroid at the commencement and at the termination of the work, and at frequent intervals between, in order to detect any irregularity in the instrument. The Aneroid is chiefly useful in working from one known elevation to another to determine the approximate heights of intermediate points. For long-continued observations, unchecked, or for long profiles, the barometer is of little or no use. The instrument should be carefully handled, and when used held in a horizontal position, in order that the counterweight may

* The outer case of the Aneroid varies from 3 to 6 inches in diameter, and from 1 to 2 inches in depth. The price ranges from 15 to 50 dollars. The cheaper ones are, however, of little value.

act properly. For nice work, an allowance should be made for variations in the temperature, both of the air and of the instrument. This has not commonly been done in using the Aneroid; though the results would certainly be more reliable if this point was regarded.*

* Long-continued observations made to compare the Aneroid with the standard barometer, have shown that with good instruments the difference rarely exceeds the hundredth of an inch. The action of the Aneroid, under a great variation of pressure, is seen by the following: Two of the largest sized and best French instruments were set alike at the Glen House, in the White Mountains, New Hampshire, both reading 28.31 inches, at 8 A. M., July 23, 1869. One of them was then taken to the top of Mount Washington, 8 miles distant, and 4657 feet higher, while the second one was left at the Glen House. The reading upon the Summit, at noon, was 23.83. At 4 o'clock P. M., at the same place, the reading was 23.81. On reaching the Glen House, at 6.55 P. M., the reading of the barometer, which was taken up the Mount, was 28.25, while that of the instrument left at the Glen was 28.32. Thus the barometers, which were set together in the morning, differed 0.07 of an inch after one of them had been taken up 4657 feet. The next morning, however, the difference was only 0.02 of an inch. The actual results obtained by the rule above and the data at Mount Washington are as follows: The reading at the Glen (28.31) and at the Summit (23.83) give the elevation 4726. The Summit reading (23.81) and the Glen reading (28.25) give 4691. But if we add 0.07 (the amount by which the barometers differed at night) to the noon reading at the Summit, making the readings at the Glen and Summit respectively 28.31 and 23.90, the elevation is 4646. Again, adding the 0.07 to the 4 P. M. reading at the Summit, we have, with the readings at top and bottom, respectively 23.88 and 28.32, the elevation 4678. The average of these last two results is 4662 feet; or only 5 feet more than the actual elevation.

CHAPTER II.

SURVEY.

Topographical Sketching.

In making a railroad survey, it becomes necessary to represent upon paper the topography of the district traversed. A complete topographical map shows everything that exists upon the ground, — the position of watercourses, the relief of the surface, the state of cultivation, the roads, the dividing lines between different owners, and the town and county boundaries. The sketches required in a railroad survey need only to include the undulations of the surface for a short distance upon each side of the centre line of the route.

The making of these sketches consists in tracing the irregular lines (contour lines) formed by the intersection of the natural surface and a series of horizontal planes, placed at a greater or less vertical distance, according to the detail required.

Suppose that we wish to represent, upon a horizontal surface, a right cone. The base, A B, Fig. 7, is shown upon the plan by the larger circle. If the elevation is cut by any horizontal plane, as D E, the intersection of that plane with the conical surface is shown upon the plan by a small circle, of which the diameter is equal to D E. In the same manner, if the cone is oblique, as in Fig. 8, the base A B and the horizontal section D E are shown on the plan by eccentric ellipses instead of by concentric circles. If we have the intersecting cones, of the same size, as in Fig. 9, the bases are shown on the plan by the outer circles cutting each other at two points ; and the sections F H and I K, by

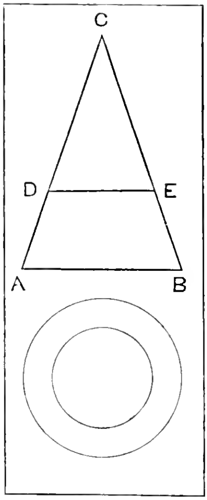


Fig. 7.

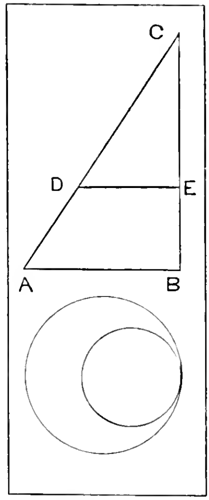


Fig. 8.

the small separated circles. If the cones are of different sizes, as in Fig. 10, the circles on the plan are also of different sizes.

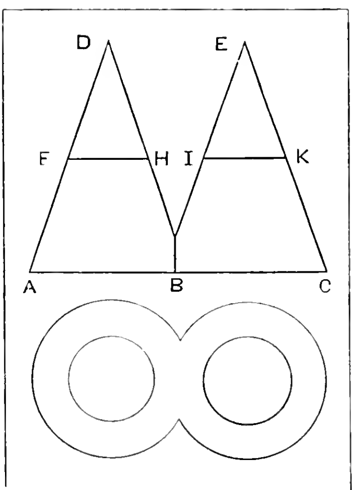


Fig. 9.

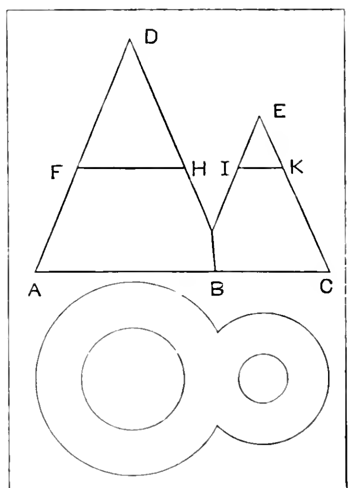


Fig. 10.

The above figures contain the elements of the whole of topographical drawing, as regards the tracing of contour lines. In the section of regular figures the horizontal projections of those figures are also regular; but in a broken surface, like the ground, the lines become quite irregular.

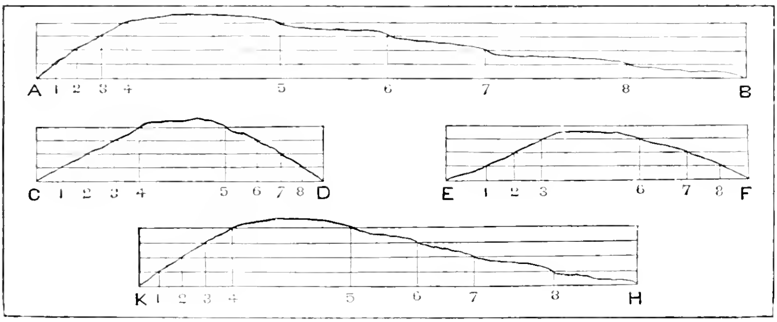


Fig. 11.

Suppose that we wish to show, upon a plan, the relief of a hill of which we have the profiles given by the irregular lines in the sections A B, C D and E F, Fig. 11. Intersect each of these

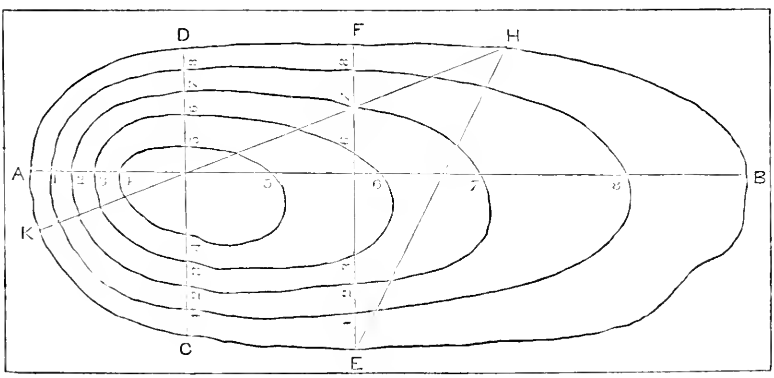


Fig. 12.

profiles by the equidistant horizontal lines shown in the figures. Upon the profile A B, drop a vertical from each of the intersec-

tions of the surface line with the horizontals, and upon the line A B, on the plan Fig. 12, make A 1, A 2, A 3, A 4, and so on to A B, equal to the same distances on the section. Next draw, on the plan, the line C D, in the same position with regard to the line A B, as those lines have upon the ground, and having found the horizontal distances C 1, C 2, C 3, etc., upon the profile C D, transfer them, in the same way as the distances upon the profile A B were transferred, to the line C D on the plan. Proceed in the same manner with the profile E F. The points A B C D E F are all upon the base lines of the several profiles, and thus at the same level; and an irregular line drawn through those points, upon the plan, will represent the intersection of the natural surface by a horizontal plane. The points 1 1 1, 8 8 8, again, are all at the same height above the base line; and thus another irregular line passing through those points, on the plan, will represent the intersection of the natural surface by another horizontal plane above the first. In the same manner the remaining contour lines upon the plan are found. When these contour lines approach near to each other we see at once that the surface is steep. When, on the other hand, the distances are large, we know that the ground falls gently. This is plainly seen upon the profiles.

Having now the topographical sketch, shown in Fig. 12, we may deduce therefrom a section or profile upon any line we choose. Thus, if we would have a profile upon the line K H, on the plan, we have only to reverse the proceeding by which we obtained the plan from the profiles A B, C D, and E F. Laying off upon a horizontal line K H, Fig. 11, the distances K 1, K 2, K 3, K 4, etc., and raising the verticals through these points, we find the profiles by drawing the irregular line through the intersections of the verticals and the corresponding horizontals. In the same manner we should obtain the profile of the line E H. Thus we see how complete a knowledge of the ground a correct topographical map gives.

Field sketches for railroad work are generally made by the

eye; the note-book being ruled in squares, by lines about a fourth of an inch apart, such squares being assumed as 100 feet to the side. When we need a more accurate sketch than this method gives, we may cross section the ground, either with rods made for the purpose, or with the level. By making a very detailed map, and filling it in with correctly drawn topography, we may, if we choose, make the location itself upon the paper, and afterwards transfer it to the ground.

In the above remarks we have dealt only with a single summit; but the mode of operation is the same when applied to a group or range of hills, or indeed to any piece of ground.

Running the Preliminary Line.

Having by the reconnoissance found approximately the place for the road, we proceed to run a trial line by compass, selecting what appears to be the best ground, staking out the centre line, and sketching in the topography right and left.

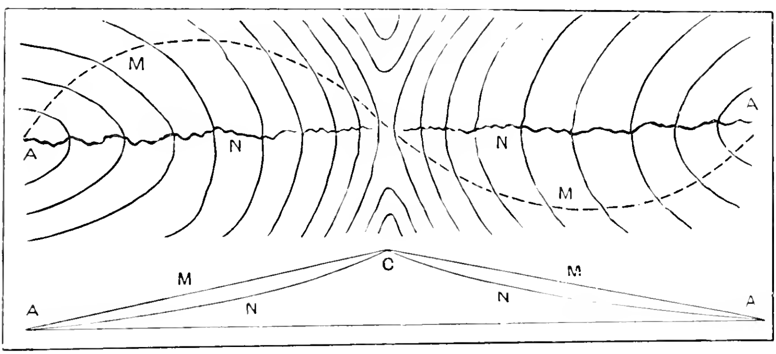


Fig. 13.

In following the bed of a stream we obtain generally a line like A N C, upon the section in Fig. 13. The lowest line of the valley, though quite moderately inclined at first, rises more and more rapidly towards the source of the stream, as shown by the closer approach of the contour lines on the plan. That the line

may ascend at a uniform rate from A to the summit, the horizontal distances between the contour lines must be equal at all parts of the ascent. This equality is effected by causing the surveyed line to cut the contours at right angles during the first part of the ascent, and obliquely as the summit is approached, and the contours become closer together. In this manner we obtain the profile A M, M A. The contour line is level; the line cutting the contour at right angles is the steepest line that the ground admits of; and as we vary between these limits we vary the inclination.

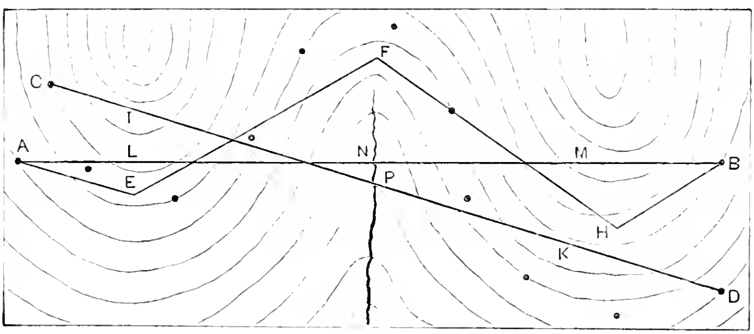


Fig. 14.

The effect upon the profile of cutting the contour lines in different directions is shown by Figs. 14 and 15. If we connect the points A and B, upon the plan, Fig. 14, by the straight line A B, we obtain the profile shown on Fig. 15, by the line A L N M B. If we follow the contour line round from A to B, we should have the horizontal line A B on the profile; and if we select a route upon the plan midway between the straight line and the contour line, as A E F H B, we shall have, as the corresponding profile, A E F H B, in Fig. 15. In connecting the points C and D, which are at different elevations, by the straight line C D, we get the profile C I P K D. If we wish to descend upon the natural surface at a uniform and given rate from C to D, knowing the rate of incline and the vertical distance between the contour lines, we get at once the corresponding horizontal dis-

tance from one contour line to the next, which applied, as by the heavy points in Fig. 14, gives the required descent, shown on the profile by the straight line from C to D. The line A E F H B, on the plan, is, of course, longer than the straight line A B;

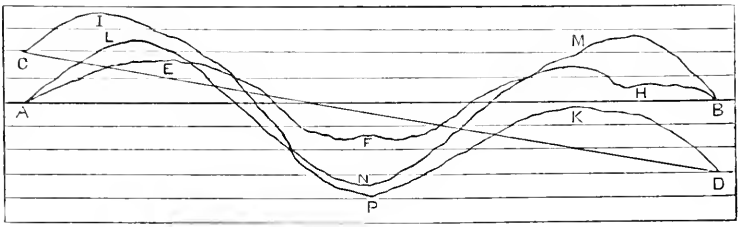


Fig. 15.

and the contour line is longer still. Such increased length is not, however, represented upon the profile, as the object has been simply to show the general relation between the plan and profile, and the use of correctly drawn contour lines in adjusting any route to the ground.

General Establishment of Grades.

In the application of grades or inclines to the establishment of a railway route, we should bear in mind that between two places, which are at the same absolute elevation, there should be as little rise and fall as possible; and that between points at different elevations we should endeavor to have no rise while descending, and consequently no fall upon the ascent. These conditions can, however, in practice, seldom be complied with exactly; but we may often approach them. In many cases we have to choose between two systems of grades — the one involving a long but gradual and uniform rise, the other a short but steep ascent, with the remainder of the line level, or nearly so. The total resistance upon the two systems, involving the same amount of ascent, will be the same; but a great difference may be made in the method employed in overcoming that resistance.

If we have a grade ten miles long, rising at the rate of 20 feet per mile, we adapt our machinery to hauling its ordinary load up that incline, and we require from it a constant expenditure of power while ascending, and on the descent it is all the way aided to a profitable extent by gravity. If, however, the first eight miles are level, and the remaining two miles ascend at the rate of 100 feet per mile, an engine to work the incline would be too heavy for the level portion; and in the descent we should have more aid from gravity upon the incline than we required, and none at all upon the level.

The effect of the arrangement in detail of the grades upon the amount of work to be done in reducing the natural surface of the ground to the finished road bed, is shown in Fig. 16. The

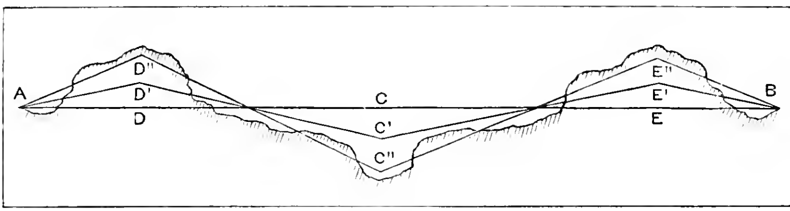


Fig. 16.

level grade from A to B involves a large amount of earth cutting at D and E, and a large amount of embankment at C. By raising the grade lines to D′ and E′, in the cuttings, and depressing the level of the embankment from C to C′, we at once reduce both the amounts of excavation and embankment; and by raising the road bed still more, to D″ and E″ in the cuttings, and depressing it to C″ upon the embankment, we reduce still more the quantity of work to be done, but at the same time we render the road harder to be operated.

CHAPTER III.

LOCATION.

Circular Curves.

The route, as furnished by the survey, consists of a series of straight lines of different lengths, forming at their connection angles of greater or less magnitude. These angles require to be rounded off, in order to render the passage easy from one straight portion of the line to another. This is done by means of curves, generally portions of a circle, which are laid out upon the ground, either by offsets from a chord or tangent, or by angles of deflection from the tangent, by means of a theodolite or a transit, the latter method being the most common. The process is simple, but requires care. Let A D and B D, Fig. 17, be two portions of a surveyed line, making an angle at D, and let it be required to connect these lines by a circular curve from A to B. If we fix the radius of curvature, the problem which first presents itself is to find the distances D A and D B, which are of course alike. If, by the nature of the ground, the points A and B are fixed, then we have the distances D A and D B, and we require to know the radius. Suppose that we have the distances A D and B D given, and the angle A D B, all of which are obtained by measurement upon the ground. D A C is of course a right angle: A D C is half of A D B; and A C D is 90° less A D C. Knowing all of the angles, and the side A D, we get the radius A C by the proportion —

sin. A C D : sin. A D C : : log. A D : log. A C ;

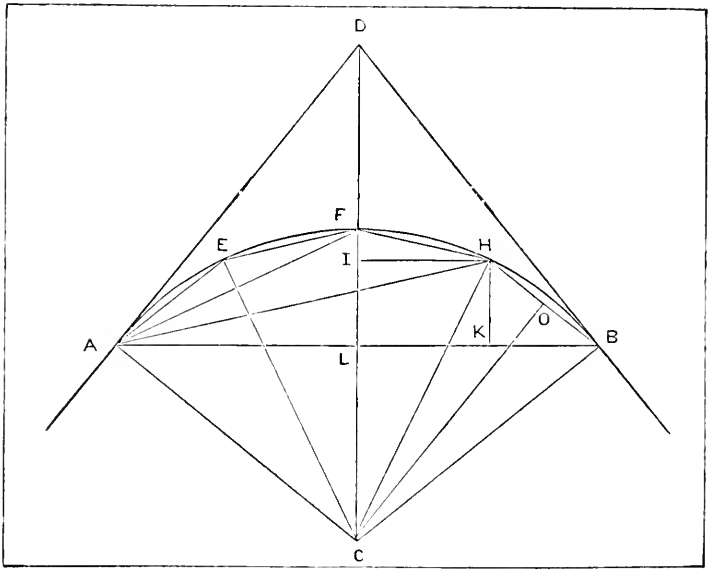


Fig. 17.

or, knowing the radius and requiring the distance —

sin. A D C : sin. A C D : : log. A C : log. A D.

If we assume the radius as 5730 feet, and the angle A D B as 78°, then A D C is 39°, and A C D is 90 — 39, or 51°, and the second proportion above becomes —

sin. 39° : sin. 51° : : log. 5730 : log. 7076.

Thus the distance from D back to A or to B, the point at which a curve of the assumed radius must commence to be tangent to the lines A D and B D at A and B, is 7076 feet.

To put the curve upon the ground, we place an angular instrument at A, and lay off the angles D A E, D A F, D A H, etc., which, combined with the distances A E, E F, F H, etc., fix successive points in the curve. To find the angle D A E, or D B H, draw first C O perpendicular to the chord B H. The angle D B H

is equal to the angle B C O, both being 90° less C B O. The angle B C O is found by the proportion —

log. B C (rad) : log. B O : : sin. 90° : sin. B C O.

The chords B H, H F, etc., are commonly taken at 100 feet, except in curves of very small radius, when they may be taken at one half or one fourth of that length. In the above formula, if we take the radius of the curve at 5730, and the chord at 100 feet, we get as the angle B C O, or D B H —

log. 5730 : log. 50 : : sin. 90° : sin. 0° 30′.

The angle subtended by the whole chord B H will be just double this, or 1°. A curve with a radius of 5730 feet is thus called a one-degree curve. A curve with a radius half as large, or 2865 feet, a two-degree curve, and so on; the degree of curvature being inversely as the radius, and given by the proportion —

rad. : 5730 : : 1° : deg. of C.

Thus, if the radius is 2000 feet, the degree of curvature, or the angle subtended at the centre by a 100 feet chord, is found by the proportion below —

2000 : 5730 : : 1° : 2°.86 or to 2° 51′.6.

And if the degree of curvature is known, and we require the radius, we get it by the proportion —

deg. of C : 1° : : 5730 : radius.

Thus, if the degree of curvature, or the angle subtended at the centre by a 100 feet chord, is 3° 30′, the proportion becomes —

3°.30′ : 1° : : 5730 : 1637.

The angle of deflection from the tangent, by which we lay out the curve, or the angle D B H, is one half of what we have termed the degree of curvature; i. e., one half of the angle subtended at the centre by the chord.

The following table shows the degrees of curvature and the corresponding radii for different curves, the chord being 100 feet : —

Degree.	Radius.	Degree.	Radius.	Degree.	Radius.
1	5730	4	1433	9	637
$1\frac{1}{4}$	4584	$4\frac{1}{4}$	1348	10	574
$1\frac{1}{2}$	3820	$4\frac{1}{2}$	1274	11	522
$1\frac{3}{4}$	3274	$4\frac{3}{4}$	1207	12	478
2	2865	5	1146	13	442
$2\frac{1}{4}$	2547	$5\frac{1}{2}$	1042	14	410
$2\frac{1}{2}$	2292	6	955	15	383
$2\frac{3}{4}$	2084	$6\frac{1}{2}$	882	16	359
3	1910	7	819	17	338
$3\frac{1}{4}$	1763	$7\frac{1}{2}$	764	18	320
$3\frac{1}{2}$	1637	8	717	19	303
$3\frac{3}{4}$	1528	$8\frac{1}{2}$	675	20	288

If we require for any purpose the versed sine, or middle ordinate, upon any chord, as F L, Fig. 17, we have only to find C L, which is $\sqrt{BC^2 - BL^2}$, and to subtract it from the radius. To find any other ordinate, as H K, at a known distance K L, from the versed sine, we have $CH^2 - HI^2 = CI^2$; and C I less C L is I L, or H K, the required ordinate. The following table gives the ordinates in feet and decimals, for different radii, at four points upon a 100 feet chord; the points being the versed sine, and three equidistant intermediate points between the centre and the end of the chord. The same three ordinates being reversed in their order, and applied upon the other side of the versed sine, we have seven points in the curve between the ends of the chord.

Rad. of Curve.		Ordinates on a 100 Feet Chord at			
Degree.	Feet.	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	V. S.
5	1146	0.48	0.82	1.02	1.09
6	955	0.57	0.98	1.23	1.31
7	819	0.67	1.15	1.43	1.53
8	717	0.76	1.31	1.64	1.75
9	637	0.86	1.47	1.84	1.96
10	574	0.96	1.64	2.05	2.18
11	522	1.05	1.80	2.25	2.40

Rad. of Curve.		Ordinates on a 100 Feet Chord at			
Degree.	Feet.	1/8	1/4	3/8	V. S.
12	478	1.15	1.97	2.46	2.62
13	442	1.24	2.13	2.66	2.84
14	410	1.34	2.30	2.87	3.06
15	383	1.44	2.46	3.07	3.28
16	359	1.53	2.62	3.28	3.50
17	338	1.63	2.79	3.48	3.72
18	320	1.73	2.96	3.69	3.93
19	303	1.82	3.12	3.90	4.15
20	288	1.92	3.29	4.10	4.37

Compound Curves.

It is often found in practice that a simple circular curve will not fit the ground in a satisfactory manner; but that the line needs to curve more sharply at one place than at another. In such cases we employ what is termed a compound curve, in which several curves of different radii are combined. Thus, in Fig. 18, to connect the tangents A′ A and B′ B we curve from A

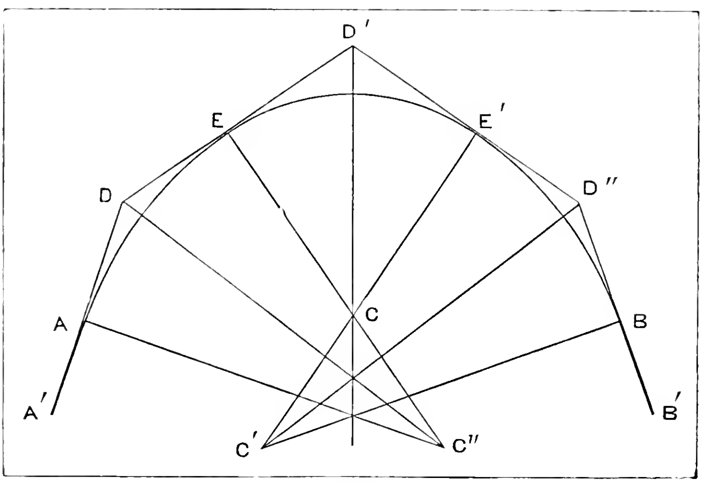


Fig. 18.

to E upon the centre C″, and radius A C″; from E to E′ upon the centre C, and radius E C; and from E′ to B upon the centre C′, and radius E′ C′. In order that the adjoining curves should be properly connected, they should have a common tangent at the point of compound curvature, as at E and E′; or, in other words, the centre of the curve from E to E′ must lie in the radius E C″ of the curve from A to E; and in the same manner the centre C′ must lie in the radius E′ C produced.

Reversed Curves.

Again, it is often necessary to reverse the direction of the curvature, as in Fig. 19. To connect A′ A with B′ B, we may com-

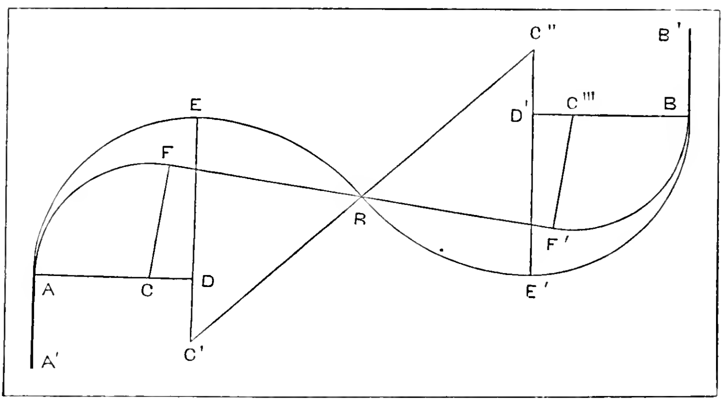


Fig. 19.

mence by curving from A to E on D as a centre, thence with C′ as a centre from E to R, the point of reversing, and from that point with the centres C″ and D′ to the tangent point B. Or we may find it best to insert a piece of straight line, or tangent, as F F′, between the curves, giving the line A F F′ B. In the first mode the curves must have a common tangent at the point of reversing; or, in other words, C′ R and C″ must lie in the same straight line.

There are many problems necessary to be understood by the practical engineer relating to the location of railway curves. For the solution of these, with ample discussions and rules, and full tables for practice, the reader is referred to the works of Trautwine and Henck.*

General Remarks upon Curves and Grades.

Upon a judicious application of grades and curves in the location of a railway the success of the undertaking in a great measure depends. We may spend so much in obtaining light grades and curves of large radius that the business of the road never can pay the interest upon the first outlay; and we may introduce grades so steep and curves so sharp that the cost of hauling trains over it may consume the whole income. Between these extremes lies that application of grades and curves which shall, without extravagant outlay, give a road that can be economically operated, and thus be at once a public improvement and a profitable investment.

In order to apply the best system of grades and curves to a projected railway, and at the same time to be able to compare the merits of any number of lines having different characteristics, it is necessary to understand what effect upon the working of a railway these elements have.

Resistance to Motion upon a Straight Line.

The resistance to the movement of a railway train upon a straight and level road depends upon a variety of conditions, such as the state of the rolling machinery and of the track, the climate, the season, and even the weather. According to Mr. D. K.

* The Field Practice of Laying out Circular Curves for Railroads. By John C. Trautwine. Civil Engineer. Field Book for Railroad Engineers, containing Formulæ for Laying out Curves, etc. By John B. Henck, A. M., C. E.

Clark,* whose experiments were made upon trains running as nearly as possible under the conditions of ordinary practice, the total resistance is given by the following rule: —

$$R = \frac{V^2}{171} + 8,$$

in which R is the resistance in lbs. per ton of the engine, tender, and train, and V the velocity in miles per hour. The rule supposes the line to be straight and level, the weather favorable, and the track and machinery in good order. Thus, at 20 miles an hour, the resistance would be: —

$$R = \frac{20 \times 20}{171} + 8 = 10.3 \text{ lbs. per ton of the whole train.}$$

Resistance due to Grades.

The resistance due to a grade of any inclination is found by the simple mechanical rule of multiplying the load by the height, and dividing the product by the length of the incline. Thus the resistance per ton due to a 24 feet grade is —

$$2240 \times \frac{24}{5280} = 10.2 \text{ lbs.},$$

or nearly the same as the resistance upon a level line at a speed of 20 miles an hour. The resistance to motion, therefore, upon a 24 feet grade is double that upon a level at a speed of 20 miles an hour; whence the power required to draw a train up one mile of 24 feet grade at 20 miles an hour, would draw it two miles upon a level at the same velocity. As the resistance upon a level, however, increases with the speed the greater the velocity of the train the steeper is the grade to require a double expenditure of power, as shown by the following table: —

* Railway Machinery. Part Fourth. See also *De la Résistance des Trains et de la Puissance des Machines.* Par Vuillemin, Guebhard, et Dieudonné. Paris, 1868.

Velocity in Miles per Hour.	Resistance in Pounds per Ton.	Grade to double the Resistance.
15	9.3	22
20	10.3	24
25	11.7	27
30	13.3	31
40	17.4	41
50	22.6	53

By the table it is seen that while a grade of 24 feet per mile doubles the resistance at 20 miles an hour, at 50 miles an hour we should require a grade of 53 feet per mile to do the same.

Equating for Grades.

Inasmuch as the total resistance offered by any incline depends upon the amount, and not the rate of ascent, we may compare lines having different systems of grades by simply making a certain allowance for each foot of vertical rise; or we may determine the number of feet of ascent which shall involve an expenditure of power equal to that required to move the train one mile on a level, and divide the whole ascent in any line by that number; the quotient being the number of miles to be added to the actual distance to get the equivalent horizontal length, or, as it is termed, the equated distance. For example: If we take the speed as 25 miles an hour, the number of feet of vertical ascent, which shall consume an amount of power sufficient to haul the train one mile on a level, is, by the preceding table, 27. If we have a line 100 miles long, involving an ascent of 540 feet, the equivalent level length would be —

$$100 + \frac{540}{27}, \text{ or } 100 + 20 = 120 \text{ miles.}$$

The above has been a common mode of equating for grades, and represents a length proportionate to the power expended. But it does not represent a length proportionate to the *cost* of exerting that power, which is what we require. Of the whole

expense of operating a railroad, a few items are directly proportional to the power exerted in hauling the trains; other items are increased, but not to so great an extent, while others are not affected at all. Thus the fuel used during the ascent of a grade may be taken as proportional to the power exerted. The wear of the rails is more upon a grade than upon a level, though to what extent can hardly be stated precisely.

General expenses, such as agents, salaries, insurance, and many others, are not affected at all.

If the grade is not over say 25 feet per mile, so that it may be worked by simply increasing the weight of the engines, without augmenting their number, we may assume one sixth part of the total expense of maintenance and operation to be doubled by doubling the power exerted; in which case, instead of adding a mile for each 27 feet of ascent (or other number according to the speed), we should add only one sixth of a mile; or we should multiply the numbers in the third column of the preceding table by six, as below:—

Speed in Miles per Hour.	Resistance in Pounds per Ton.	Grade to double the Resistance.	Rise in Feet to double the Cost of working.
15	9.3	22	132
20	10.3	24	144
25	11.7	27	162
30	13.3	31	186
40	17.4	41	246
50	22.6	53	318

Thus, at a speed of 25 miles an hour, for each 27 feet of ascent we shall consume an amount of power sufficient to move the train one mile upon a level; but to consume an *expense* sufficient to maintain and operate one mile, we must ascend six times the above amount, or 162 feet.

When the grade becomes so steep as to demand an additional number of engines, the expense is increased more than by the amount stated above; and therefore we should multiply the num-

bers in the third column of the foregoing table by a less number than six. Probably for grades from 25 to 50 feet *four* would be a sufficiently large multiplier; from 50 to 75 feet, *three;* and for grades from 75 to 100 feet, *two:* so that we may form the following table for an average speed of 25 miles an hour:—

Grades.	Equating Nos.
0 to 25	27 × 6 or 162
25 to 50	27 × 4 or 108
50 to 75	27 × 3 or 81
75 to 100	27 × 2 or 54

In descending, the grade, instead of being an obstacle, becomes an aid; as the train will roll down the incline of its own accord, if it is steep enough. The momentum, too, acquired during the descent, becomes useful in helping the train after the foot of the grade is reached, to some slight extent. The available momentum depends upon the speed of the train when it arrives at the foot of the incline, and not at all upon the length or total amount of the fall which has been traversed. As soon as the grade becomes steep enough to let the train roll down of itself, we make a saving in the amount of fuel used in the engine; and if the grade is ten times steeper we do no more. Thus, in making a reduction on account of descending grades, in order to find the equivalent level length, we do not reverse the process by which we found the addition to be made for ascending, as the gain from any descent does not depend upon the total amount of fall. If 25 feet per mile is enough to allow a train to descend by gravity, using only steam enough for lubrication of the cylinders, for every mile of such descent we save the fuel which would haul the train one mile upon a level; and as about one eighth of the expense of operation is chargeable to fuel, we may deduct therefore one eighth of a mile from the measured length; and for every mile of grade descending at a less rate, a proportionate amount; but for any excess over 25 feet per mile no allowance should be made. Thus, if we descend a mile upon a 40 feet

grade, we may deduct an eighth of a mile for 25 out of the 40 feet, and throw aside the remaining 15. While, however, we save in fuel upon the descending grade, we have a greater wear and tear of track and machinery; so that, all things considered, we should make no reduction at all for such descent. Indeed, when the grade is very steep, we probably lose more by the extra wear and tear than we gain in the saving of fuel.

To illustrate the above remarks, suppose that we have surveyed two lines, having the following characteristics:—

LINE A B, 100 MILES.	LINE C D, 96 MILES.
Grades. Going from A *to* B.	*Grades. Going from* C *to* D.
30 miles, rising 50 feet per mile.	20 miles, rising 60 feet per mile.
20 miles, falling 20 feet per mile.	10 miles, falling 20 feet per mile.
10 miles, rising 60 feet per mile.	15 miles, rising 30 feet per mile.
15 miles, falling 15 feet per mile.	15 miles, falling 15 feet per mile.
15 miles, rising 20 feet per mile.	26 miles, rising 30 feet per mile.
10 miles, falling 40 feet per mile.	10 miles, falling 50 feet per mile,

Assuming the speed as 25 miles an hour, the numbers by which to equate are 162, 108, and 81, and we obtain the following:—

Line A B. *Going from* A *to* B.

$$100 + \left(\frac{30 \times 50}{108} + \frac{10 \times 60}{81} + \frac{15 \times 20}{162}\right) = 123.15.$$

Line A B. *Going from* B *to* A.

$$100 + \left(\frac{10 \times 40}{108} + \frac{15 \times 15}{162} + \frac{20 \times 20}{162}\right) = 107.56.$$

Line C D. *Going from* C *to* D.

$$96 + \left(\frac{20 \times 60}{81} + \frac{15 \times 30}{108} + \frac{26 \times 30}{108}\right) = 122.20.$$

Line C D. *Going from* D *to* C.

$$96 + \left(\frac{10 \times 50}{108} + \frac{15 \times 15}{162} + \frac{10 \times 20}{162}\right) = 103.25.$$

The mean equated length of A B is thus 115.35 miles, and the mean equated length of C D is 112.72 miles.

Much difference of opinion exists among engineers as to the utility of any rule for equating for grades. So much depends in practice upon the rate of ascent, upon the disposition of grades with regard to the direction of the traffic, and the adaptation of the motive power, that it is plain that no mere rule, however correctly established, can be empirically applied to the various problems occurring in railroad location. It is, however, equally plain that the amount of power employed must bear a fixed and exact relation to the amount of ascent and descent upon the road, and, therefore, in the hands of an engineer who is able to appreciate the various elements in the important problem of the best arrangement of grades, a rule for equating will be of service.

Ruling Grades.

The steepest grade upon a line is not necessarily the ruling or controlling grade. The maximum upon the Pennsylvania Railroad, on the eastern slope of the Alleghanies, from Altoona to the Summit at Gallitzin, is 95 feet per mile; while the maximum upon the western slope, from Conemaugh to the Summit, is only 53 feet per mile. This latter grade, however, is opposed to the heavily loaded freight trains going eastward, while the 95 feet grade is opposed to the lighter trains going westward. The total resistance (calling that upon a level 10 pounds per ton) upon the 95 feet grade is 50 pounds per ton; and that upon the 53 feet grade, 32 pounds. If the tractive power of the engine was 5000 pounds, it would take 157 tons up the 53 feet grade, and 100 tons up the grade of 95 feet; or the same power that would take 10 cars, each loaded with 5.7 tons, up the western slope, would take back the same train empty up the steeper eastern slope.

If the grades upon any road can be arranged so as to confine the steep ascents to one part of the line, we may work it in separate divisions, and proportion the engines upon each division, according to its ruling grade. The *length*, also, of a grade has to be regarded; as an engine intended for ordinary work will take

a train over a short incline, while the same incline continued for a great length would be beyond the steam producing power of the engine, though the adhesion might be ample.

In passing a high dividing ridge, where the traffic is to preponderate greatly in one direction, we should endeavor to ascend that slope of the ridge *up which* the heavy trains must go by a long development, giving easy grades, while we may descend upon the opposite side more abruptly. If the traffic is to be about equal in both directions, we should, as nearly as possible, make the ruling grades upon the two slopes alike. The right disposition of grades is much more essential upon a road doing a regular and heavy freight business, than upon a road devoted to passenger and light mixed trains.

Resistance from Curvature.

The resistance opposed by curves to the motion of a railway train depends upon the length of the radius, the gauge of the road, the elevation of the outer rail, the speed and length of the train, the size and arrangement of the wheels, and the form of the tread, or rolling surface, of the tires. The coning of the wheels, the elevation of the outer rail, and the examination of the resistances in detail, will be considered in the chapters upon Superstructure and Equipment.

We have here only to ascertain, as nearly as may be, the general effect of curvature upon the resistance to motion, in order to obtain some guide for comparing lines with different systems of curves; in other words, to be able to equate for curvature. The facts observed, showing the resistance from the above cause, are quite discordant, owing to the great variety in the conditions under which experiments have been made.*

* The experiments made to determine the effect of curves, so far as we have been able to obtain them, are given in the Appendix.

Equating for Curvature.

In equating for grades, we found, first, the ascent consuming an amount of power sufficient to haul a train one mile upon a level, or the grade of double resistance. So, too, in equating for curves, we require to know how much curvature will consume an amount of power sufficient to haul a train one mile upon a straight line. It is assumed, — and for our purpose here the assumption is probably correct, — that the resistance from curvature is inversely as the radius; that is, we meet with the same resistance from curvature in running one mile of 2 degree curve, as in running two miles of 1 degree curve. The number of degrees of deflection is the same in both cases. The total resistance depends upon the whole number of degrees traversed, and is independent of the radius, or length of the curve.*

The average of numerous experiments would seem to show that the resistance upon a 10 degree curve, or a curve of 574 feet radius, at a speed of 20 miles an hour, is double that upon a straight line. In traversing, therefore, a 10 degree curve, a mile long, we should consume an amount of power sufficient to haul a train two miles upon a straight line. The length of a 10 degree curve is, however, only $574 \times 2 \times 3.1416$ or 3606 feet; and this, being a whole circle, contains 360°. The proportionate number of degrees in a mile, or 5280 feet, is 527; which is thus the number of degrees, whatever the radius, consuming an amount of power which would haul a train one mile on a straight and level road at 20 miles an hour; and this is, therefore, the equa-

* The above conclusions are, of course, confined to the theoretical aspect of the question. As a matter of practice, we are much less concerned with the total than with the momentary resistance. We may distribute 1000 degrees of curvature in such a manner over 100 miles of road that it shall be practically inappreciable: while, if the same total amount of deflection was put into a number of sharp, reversed curves, they would affect the capacity of the road to a very material extent. In equating for curvature therefore, as in the case of grades, any rule for proceeding must be employed under the guidance of experience and common sense.

ting number for comparing the curvature upon different lines, just as 24 feet was the equating number for the comparison of grades at the same speed. But, as in the case of grades, a double expenditure of power does not involve a double cost. We, however, increase the cost of operation more in doubling the resistance by curvature than we do in doubling it by grades, since the effect of curvature upon the wear and tear of the engines, cars, and track is greater than that of grades. Taking the operation of the 1500 miles of railway in Massachusetts as a basis, and adding, for a double expenditure of power, demanded by curves, 25 per cent. to the cost of repairs of the roadway, engines, and cars, and 100 per cent. to the cost of fuel, we shall increase the whole expense of operating and maintaining the road by about 25 per cent. If, therefore, a mile of road containing 527 degrees of curvature demands the exertion of double the power required upon an equal length of straight line, and if the exertion of a double power involves 25 per cent. more expense, the number of degrees consuming an amount of *money* sufficient to operate and maintain one mile of road will be $\frac{100}{25}$ of 527, or 2108 degrees; which is thus the equating number for curvature at a speed of 20 miles an hour.

This number, however, being based upon a double resistance, will vary according to the actual resistance upon a straight line, and thus according to the speed, as shown in the following table, where column 3 gives the radius of the curve upon which the resistance is double that upon a straight line, these radii being made inversely to the resistances in column 2. The number of degrees of deflection in column 4 are found by the proportion —

$$\text{rad. (col. 3)} \times 3.1416 \times 2 \text{ to } 5280 \text{ as } 360° \text{ to No. in col. } 4;$$

and the numbers in column 5 are $\frac{100}{25}$ of those in column 4, and may be used as the equating numbers for curvature.

Speed in Miles per Hour.	Resistance in Lbs. per Ton.	Rad. of Curve of double Resistance.	Corresponding No. of Degrees in a Mile.	Equating No. in Degrees.
15	9.3	636	476	1904
20	10.3	574	527	2108
25	11.7	506	598	2392
30	13.3	444	682	2728
40	17.4	340	890	3560
50	22.6	261	1159	4636

Inasmuch as the expense of operation is more increased by sharp than by easy curvature, just as it is more increased by steep than by light grades, we should vary the equating number, in any comparison of surveyed lines, as we varied the equating number for grades in the example upon a preceding page. It is, however, impossible to say, with any exactness, what this variation should be, since we have no means of knowing what effect the sharpening of the curvature has upon the working expenses. The general effect is, of course, to make the equating number smaller for sharp curves, and larger for curves of large radius.

Suppose we have surveyed two lines, the first being 100 miles long, and having 4216 degrees of curvature, and the second being 98 miles long, and having 8432 degrees of curvature. At a speed of 20 miles an hour the equating number is 2108 degrees, and the equated distances —

$$100 + \frac{4216}{2108}, \text{ or } 102 \text{ miles};$$

$$\text{and } 98 + \frac{8432}{2108}, \text{ or } 102 \text{ miles.}$$

If we assume the cost of operation to be as the equated length, we may compare any number of routes, by adding in each case the cost of construction to the operating expense of the equated length, capitalized. Thus, supposing that we have two routes, one 100 miles in length, with an equated length of 105 miles, and another 90 miles long, with an equated length of 125 miles, the cost per mile of the first being $40,000, and of the second $60,000, and the annual operating and maintaining expenses $4000 per equated mile, we shall have the following comparison : —

$$100 \times 40{,}000 + 105 \times \frac{100}{6} 4000 = \$10{,}999{,}930;$$

$$90 \times 60{,}000 + 125 \times \frac{100}{6} 4000 = \$13{,}733{,}250.$$

Relative Resistance from Grades and Curves.

In arranging the *grades* upon any route, we may often so oppose the lightest ascents to the heaviest traffic that the resistance shall be the same in both directions; but a *curve* does not admit of being adjusted to a traffic preponderating in one direction, since the resistance is the same which ever way we traverse it. We may in some cases combine the sharper curves with the easier grades, and the larger curves with the steeper grades, so as to establish a somewhat uniform maximum or ruling resistance upon the road; but generally the same natural features that demand steep grades require at the same place sharp curves.

If we would make the resistance upon any system of grades and curves uniform, where a curve occurs upon a grade we should flatten the latter to an amount sufficient to compensate for the resistance caused by the curve. We have supposed a 10 degree curve to cause a resistance of 10 pounds per ton. But a grade of 24 feet per mile causes also a resistance of 10 lbs. per ton. Therefore, 24 feet per mile, or 2.4 feet per mile per degree of curvature is the necessary flattening of the grade that the resistance may be uniform. The following table shows the amount of flattening required upon grades for curves of different radii, the speed being 20 miles an hour:—

Radius of Curve in Feet.	Reduction of the Grade in Feet per Mile.	Radius of Curve in Feet.	Reduction of the Grade in Feet per Mile.	Radius of Curve in Feet.	Reduction of the Grade in Feet per Mile.
5730	2.4	819	16.8	442	31.2
2865	4.8	717	19.2	410	33.6
1910	7.2	637	21.6	383	36.0
1433	9.6	574	24.0	359	38.4
1146	12.0	522	26.4	338	40.8
955	14.4	478	28.8	320	43.2

At different speeds the numbers would of course vary, as the resistance depends upon the velocity. Thus, the flattening cannot be exactly adapted to all classes of trains upon a road; but it should be made as nearly as possible to suit the average requirement. Upon the Pennsylvania Road the reduction of grades upon curves has been at the rate of 1.5 feet per mile for each degree of the curve; or, say 9 feet per mile for a 6° curve. The grades upon the Central Pacific Railroad are reduced from 2 to 2.5 feet per mile per degree of curvature upon curves of from 637 to 2865 feet radii.

Upon the Baltimore and Ohio Railroad the 116 feet grade was reduced to 110 feet upon curves of 600 feet radius, and increased to 121 feet on curves of over 1000 feet radius. Upon Mr. Ellets' "Mountain Top Track" in Virginia, it was found that with a six-wheeled connected tank-engine, weighing 50,000 pounds, a difference of 58 feet per mile between the grade adopted on a straight line and that used upon curves of 300 feet radius was not sufficient to compensate for the increased resistance due to curvature. The result, however, with an engine better adapted to traversing sharp curves would be different. The immense eight-wheeled connected 40 ton engines upon the Lehigh Valley Railway, with the Bissell truck, are found to traverse sharp curves with great ease.

Financial Effect of Grades and Curves.

From what has been said, it may be seen how important it is to guard against the introduction of grades and curves without carefully considering their cost. In the preceding, we have regarded grades and curves only as demanding a greater locomotive power, and as causing an increased wear and tear of track and machinery. When, however, a road is liable to be worked up to its full capacity, any reduction of this capacity by grades or curves becomes a much more serious matter. The capacity of a road being limited by the number and weight of trains that can be run over it, if by increasing the resistance by grades or curves,

the trains are reduced in weight one half, the capacity of the road is reduced by the same amount, and the cost of transportation is doubled. When the Pennsylvania Road was built, a mile of distance saved was reckoned worth $53,000, or $10 a foot. If a road costs $40,000 a mile, and if the cost of maintenance and operation is $10,000 a mile, we might spend $206,666 to shorten the line a mile; or, according to Mr. Haupt's method, if the expenses which increase with the length of the road amount to a dollar per train mile, with 10,000 trains a year we should employ a capital of $166,666; with 20,000 trains a year, a capital of $333,333, and so on: showing a very rapid increase in the amount to be spent in shortening a line as the amount of work to be done increases. In estimating the amount to be spent in reducing grades or curves, we are of course to regard the effect of these elements upon the cost of operation in the same manner as above stated in the case of simple distance; but the interest upon the cost of construction, which applies to distance, does not apply to grades or curves. Thus, while a certain number of feet of ascent, or of degrees of curvature, may be regarded as equivalent to a mile of distance, in the matter of operation, they are less objectionable by the amount of *interest* upon the cost of building a mile of road.*

It may not, though, be advisable, in most cases certainly *is not* advisable, to make so great an outlay at the commencement of construction as the above figures would indicate. After a road has been brought into a good condition, when the traffic has become

* Upon the Pennsylvania Road a degree of curvature was valued at $50, or $18,000 for a circle of 360°. Mr. Trautwine, in his work upon railroad curves, remarks, that if the annual expenses of a road are $3000 a mile, $2000 a mile, or 38 cents a foot, will be nearly in proportion to the motive power expended. He concludes further that 1° of curvature incurs an expense equal to 8.7 feet of straight line, and thus that we may spend $38 \times 8.7 \times \frac{100}{6}$, or $55 to save 1° of curvature. The above sums, however, are based upon a very limited amount of traffic. The value of a mile of distance saved upon the Pennsylvania Road, at first assumed to be $53,000, has come to be valued, under the present traffic, at $433,000, and the value of grades and curvature will of course also increase.

well established, and attention is given to obtaining the greatest economy of operation, the minor faults of location, which, with a new and rough road and an undeveloped business were not appreciated, make themselves felt, and thus point out the way for bringing the line into a state of greater efficiency.

Examples of actual Location. — The Baltimore and Ohio Railroad.

As examples of actual location, we give in Plate I. portions of the Baltimore and Ohio, and of the Pennsylvania Railroads, both of which cross the Alleghany Mountains. The Baltimore and Ohio Railroad passes first from Baltimore to the Patapsco River, at the Relay House, thence it ascends the Patapsco, crosses to the Monocacy, and reaches the Potomac at the Point of Rocks, ascending the river upon the north bank to Harper's Ferry, 81 miles from Baltimore. From Harper's Ferry the road passes to Martinsburg, and comes again on to the Potomac near the North Mountain, following the river upon the south bank nearly to Cumberland, where it crosses and ascends upon the north bank, through Cumberland to Piedmont, at the eastern base of the mountains, 1108 feet above tide, and 206 miles from Baltimore. At Piedmont, I upon the map, the road leaves the Potomac, II, and ascends Savage River, III, and Crab Tree Creek, IV, to Wilson's Summit, V, 2620 feet above tide. From Wilson's Summit the line descends the Little Youghiogheny, VI, and crossing the Youghiogheny, VII, it ascends the same and Snowy Creek, VIII, to the Cranberry Summit, IX, 2550 feet above tide. From this point it descends Spruce Run, X, and Salt Lick Creek, XI, to Cheat River, XII, which it crosses and ascends to Cassiday's Summit. Thence it passes by the head-waters of Pringles' Run, to the Kingwood Tunnel, XIII, 1820 feet above tide ; from which it descends by Raccoon Run, XIV, and Three Fork Creek, XV, to Tygart River, at Grafton, XVI, 279 miles from Baltimore, and 1008 feet above the sea. From Grafton the road descends the

valley of Tygart River to the Monongahela at Fairmount. From Fairmount the line runs in a northwesterly direction to Wheeling, upon the Ohio River, 379 miles from Baltimore. The difficult portion of this road lies between Piedmont and Grafton; the former place lying at the eastern, and the latter at the western base of the mountains. It is this mountain division that is represented in Plate I., Fig. 1, which is reduced from the original manuscript map in the company's office at Baltimore. The grades upon this section are very heavy. The road ascends in the upper part of Raccoon Run, going from west to east at the rate of 105.6 feet per mile, for about 5 miles, to the western end of the Kingwood Tunnel. From Cassiday's Summit, about two miles east of the Kingwood Tunnel, the road descends the slopes of Cheat River, at 105.6 feet per mile, for about 4 miles. It then ascends Salt Lick Creek and Spruce Run for about 10 miles, at 116 feet per mile, to the Cranberry Summit. Thence for about 20 miles the grades are undulating, and comparatively light, until Wilson's Summit is reached, from which point the road descends Crab Tree Creek and Savage River, falling about 1500 feet in 16 miles, or at 116 feet per mile for a large part of the distance. The sharpest curves upon this line have a radius of 600 feet. Inspection of the map will show how closely the road from Piedmont to Grafton follows the watercourses. The selection of the route was a matter of great difficulty, and the location was, for the time of its establishment, the boldest piece of railway engineering in the world. The grades opposed to the traffic are very heavy; the freight from the west being obliged to ascend long grades of 105 and 116 feet per mile; yet the large quantity of freight transported, and the low price of fuel upon the line, enable the work to be done economically. A A A, upon the map, is the Front Ridge of the Alleghany Mountains; D D D, the Great Back Bone Mountain; S, Savage Mountain; F, Folly Mountain; C C, Cranberry Mountain; B B, Briery Mountain; and L L L, Laurel Hill.

The Pennsylvania Railroad.

The Great Pennsylvania Railroad passes first through Chester and Lancaster counties from Philadelphia to Harrisburg. From Harrisburg it ascends upon the east bank of the Susquehanna about 5 miles, crosses the river, and follows the west bank 9 miles, to Duncannon. From Duncannon it ascends the Juniata and the Little Juniata to Altoona, at the eastern base of the Alleghanies, 237 miles from Philadelphia, and 1171 feet above tide. From Altoona the road ascends the eastern slope of the main ridge of the Alleghanies, as shown in Plate I., Fig. 2, working up the side hills by long developments, and passing through the Alleghany Tunnel, it reaches the summit level at Gallitzin, 249 miles from Philadelphia, and 2154 feet above high tide in the Schuylkill River. From Gallitzin it crosses the head of the Clearfield waters, and strikes the Little Conemaugh, which it follows closely to Johnstown. From Johnstown the line follows the south bank of the Conemaugh through Laurel Hill and Chestnut Ridge, and doubling around the latter, follows its western slope, crosses the Loyalhanna at Latrobe, ascends to George's Summit, and descends to Pittsburg, at the junction of the Alleghany and Monongahela, 354 miles from Philadelphia.

The passage of the above road across the Alleghanies is shown in Plate I., Fig. 2. The rate of ascent going west, from 1.5 miles above Altoona to the Summit, is 931 feet in 10.5 miles; the steepest grade being 95 feet per mile. The descent from the Summit to Conemaugh is 934 feet in 24.5 miles; the maximum grade being 53 feet per mile. The 95 feet grade is opposed to the lighter trains going west; while the easier grades upon the western slope are opposed to the heavily loaded freight trains moving east.

The region represented in the Fig. is very interesting in an engineering point of view. M M M is the main ridge of the Alleghanies; J J, the Little Juniata; A, Altoona; H, Hollidaysburg; S, the Summit; N N, the Clearfield waters; and C C C, the

Little Conemaugh. Besides the main line of the Pennsylvania Railroad, P P, there is a branch to Hollidaysburg, and from that place a railway built by the State, but afterwards purchased by the Pennsylvania Railroad Company, and not operated at present, which ascends by the development B B B B, and passing through the Summit, connects with the main line at Gallitzin. In addition to the above routes the old Alleghany Portage Railroad, built in order to transfer canal freight from Hollidaysburg to Johnstown, crosses the mountains abruptly by the route E E E E. This passage of the main ridge was effected by means of ten inclined planes, five upon each side of the Summit, varying in inclination from 7.25 to 10.25 feet in 100. Still another route crosses the Alleghanies at the same place, viz., the common road from Hollidaysburg to Ebensburg, shown by the line D D D D.

Connellsville and Southern Pennsylvania Railroad.

As a most instructive illustration of topographical sketching, and of railroad location upon difficult ground, we give in Plate I., Fig. 3, a plan of a portion of the route selected for the Connellsville and Southern Pennsylvania Railroad, about 25 miles west of Bedford, which, though not constructed, serves to illustrate many of the preceding remarks. During the location of this road, in 1865 and 1866, after ascending the western slope of the Alleghanies in Somerset county, running up Buffalo Creek, and crossing the main ridge at Dieter's Summit, some difficulty was experienced in descending the eastern side of the mountain; and it was found necessary to obtain the required length by running the line out upon one of the spurs, following down the southern slope for about 4000 feet, then curving towards the spur and tunnelling under it, coming out upon the opposite side, and descending upon the northern slope for about 3000 feet, and finally curving again towards the spur, tunnelling under it a second time, and under the higher part of the line itself, and coming out again upon the southern slope, by which the line

descends into the valley of Three Lick Creek, a tributary of the Raystown Branch of the Juniata.*

The summit of the ridge is shown in the plate by S S S S. The scale is shown upon the engraving. The contour lines are ten feet apart vertically. The radius of curvature at the upper or right hand tunnel is 717 feet, and that at the lower or left hand one 819 feet. It will be an instructive example for the engineering student to make the profile of the natural surface along the located line, from the contour lines in the engraving, and to determine the rate and amount of descent accomplished.

Union Pacific Railroad.

Plate II. represents the location of the Union Pacific Railroad, at a point where some difference of opinion existed as to the best route to be adopted.

Quaking Asp Mountain, of which a portion of the eastern slope is shown in the plate, forms a part of that ridge which extends from the Uintah Mountains northward along the eastern rim of the Great Basin, and separates the waters flowing into the Great Salt Lake from those flowing into the Pacific. Upon the western slope of this ridge rise small tributaries of Bear River, the longest of which, Sulphur Creek, presented the most favorable means of ingress to the valley. From the eastern slope spring the head waters of Ham's and Black's Forks of Green River, which latter finds an outlet by means of the Colorado into the Gulf of California. A small tributary of Black's Fork, Muddy Creek, was found to give the best approach from the east to the summit of the Ridge. Aspen Summit thus forms the dividing ridge between the waters of Sulphur Creek on the west side, and of Milk Creek, a small branch of Muddy, on the east.

* The chief engineer of the above location was R. B. Lewis, Esq. The plate has been made by means of a tracing from the original plan, obtained through the kindness of the Engineer Department of the Pennsylvania Railroad Company.

It is crossed at an elevation of 7546 feet above tide; being the highest point but one reached by the Pacific Railroad.*

Plate II. embraces the line from the east end of Aspen Summit cut to a point $7\frac{3}{4}$ miles east of Piedmont (P), the distance upon an air line being 9.28 miles, and by the road as built, and represented by the full black line, 14.68 miles.

The difference of level between the points A and F is 702 feet. The maximum grade upon the line as built is 60.7 feet per mile. It having been suggested, after the completion of the work, that a change in location which should shorten the line and reduce the amount of curvature would improve the road, careful surveys were made, as shown by the dotted lines, one A B S D E F, the Soda Spring Summit line, having a length of 10.59 miles, and grades of 105 and 116 feet per mile; and another, the New Piedmont Connection, being simply a modification of the upper part of the line as built, from A through B to C, having also a grade of 116 feet per mile. The relative lengths, and the amount and arrangements of grades, are shown upon the profiles. The line as built has a remarkably uniform ascent, with an average grade of 48 feet per mile, and a maximum of 60.7 feet, with no descending grades upon the ascent. On the other hand it is 14.68 miles long, and has 1226° of curvature, against 10.59 miles of length, and 647° of curvature, with an average grade of 66 feet, and a maximum of 116 upon the Soda Spring Summit line. The lines compare generally as below: —

	Line as built and operated.	Soda Spring Summit Line.	Piedmont Connection.
Length (miles),	14.68	10.59	13.32
Rise (feet),	702	702	702
Fall (feet),	0	30	14
Average grade,	47.8	66.3	52.7
Max. grade,	60.7	116.0	116.0
Curvature,	1226°	647°	601°

* The summit at Sherman, in the Black Hills, is 8235 feet above tide water; the main ridge of the Rocky Mountains, near Creston, 7100, and the Sierra Nevada, at the Donner Pass, 7017 feet.

Notwithstanding the saving in distance and in curvature by the second and third lines, and the farther advantage of reducing very much the protection required from snow, the abrupt grades were judged to overbalance all such gains; and the line as built was decided by the engineer of the road to be the best.

Central Pacific Railroad.

Plate III. represents a portion of the location of the Central Pacific Railway across the Sierra Nevada of California. From the crossing of Donner Creek, at the eastern foot of the mountains, 6025 feet above tide, the line ascends to the summit at the Donner Pass 7017 above tide, or 992 feet in 13 miles, or an average of 76.3 feet per mile. For 11 miles, however, the grade varies from 80 to 100 feet, with a maximum for a few short distances of 105.6 feet. The curvature is generally easy; but as seen upon the plan, with a few sharp doublings of 9°, $9\frac{1}{2}$°, and 10° curves. The grade is flattened upon curves from 2 to $2\frac{1}{2}$ feet per mile per degree of curvature; i.e., the grade upon a ten degree curve would be flattened from 20 to 25 feet per mile. Within the above distance there are 8 tunnels, including that at the Summit. The latter, No. 6, is 1659 feet long. No. 13 is about 800 feet, but the remainder are quite short.* The northeastern slope of Donner Mountain, along which the line passes, is very steep, in many places precipitous, the transverse slope varying from 10° to 60°, and even steeper, while the work is for several miles from the Summit entirely through granite. Donner Peak is about 1500 feet above the line at the nearest point, while the lake is 1000 feet lower than the line at Tunnel No. 12 (see Plate). The distance from the lower end of Tunnel No. 13 across the valley of Coldstream Cañon to the line upon the opposite side is but 2000 feet, while by the line of the road it is 4.4 miles;

* See Van Nostrand's Magazine, April, 1870, for abstract of paper read January 5, 1870, by John R. Gilliss, C. E., upon Tunnels of Central and Union Pacific Railroads.

the descent between the two points being 300 feet. The work upon these steep side hills is very heavy; cuttings, which on the profile of the centre line show only from 5 to 10 feet in depth, frequently reaching from 60 to 80 feet on the upper slope; the same increase being also seen in the embankments.

Portland and Ogdensburg Railroad.

An interesting problem in railway location has been presented in tracing a preliminary line for the Portland and Ogdensburg Railroad through the Crawford Notch of the White Mountains in New Hampshire. This road, passing from Portland through Fryeburg to North Conway upon the Saco, and thence up that river, meets with no obstacle until it reaches Sawyer's River, a tributary entering the Saco from the west, at a point $10\frac{1}{2}$ miles below the "Gate of the Notch." From Sawyer's River to the Willey House, a distance of about $7\frac{3}{4}$ miles, the bottom of the valley rises at the rate of a little less than 60 feet per mile. From the Willey House to the Summit, a distance of about $2\frac{3}{4}$ miles, the river rises about 600 feet, or upwards of 200 feet per mile; but this rise is very unequally distributed, being much more abrupt near the "Gate" than lower down. The Saco valley, from Sawyer's River to the Notch, is bounded by mountains, becoming higher and more abrupt as the stream is ascended, until at the Willey House it is entirely closed in by Mount Webster on one side and the Willey Mountain on the other, the rugged slopes of which almost meet at the bottom. The river, generally harmless, is liable at any time to sudden and violent freshets which sweep every thing before them. The slopes of Mount Webster and of the Willey Mountain are, at the upper portions, of solid rock, but very precipitous. The lower slopes are composed in great part of immense masses of debris thrown down from the cliffs above. The building of a road in the bottom of the valley, from Sawyer's River to the Willey House, at a sufficient elevation to be above freshets was

shown, by careful survey, to be an operation involving neither expensive work nor excessive grades. From the Willey House to the Summit, however, it becomes necessary to place the line upon the lower slopes of the mountains, in order to secure a uniform distribution of the rise. Upon the above general plan, viz., a valley line from Sawyer's River to the Willey House, and a slope line from the Willey House to the "Gate," very careful instrumental examinations were made, and upon the plan thus prepared, the *preliminary* location, shown by the full line in Plate IV., was projected.

It being desirable to obtain the utmost economy in first cost, and also the greatest economy of operation in the completed road, and the form of the valley allowing of various systems of grades, it was suggested by the chief engineer, Mr. Anderson, that the company should avail themselves of the large experience in difficult railway location of B. H. Latrobe, Esq. Accordingly Mr. Latrobe examined carefully the plans, profiles, and the ground, and made an elaborate report to the company, from which the following abstract is prepared: —

The first question presenting itself is, *By what grade shall the Summit be approached?* this question being confined to the line between Sawyer's River and the Notch. Assuming stations 20 and 540, the first just above the Gate of the Notch, and the second a little above Sawyer's River, as the initial and terminal points, the elevations of which are respectively 1890 and 902 feet above tide, we have then to rise 988 feet in $9\frac{85}{100}$ miles, giving a grade of $100\frac{32}{100}$ feet per mile, which is thus the lowest grade that can be had, except so far as the necessary curvature shall lengthen the line, and thus reduce the rate of ascent. A line of uniform grade lying, however, as it would altogether upon the mountain slopes, would necessarily be quite an expensive one, and it thus becomes us to inquire whether a higher maximum inclination should not be adopted in order to reduce the amount of capital invested, and so, perhaps, more than compensate for the increased cost of working the line, which last we may assume

would be at its minimum with the uniform grade. Hence the necessity of tracing several lines to furnish the data by which we may compare the actual capital, or first cost, with the capital necessary to maintain and operate the road. The running of grade lines on the mountain slopes being a tedious and expensive operation, it is suggested that some mean grade be selected and put upon the ground as a base line, from which cross sections being made, the effect of moving the line up hill or down may be seen. The power required, as far as gravity is concerned, to lift the train up the 988 feet, is the same for all grades; but the resistance from friction will depend upon the relative length and upon the amount of curvature. The grade of 116 feet per mile having been for a long time safely and successfully employed upon the Baltimore and Ohio Railroad, it is suggested that that grade be selected and run down upon both sides of the valley, as a base line (from which modifications may be made in either direction) from station 20, at the Summit, until it could be reduced to such lesser grade as would best suit the ground, and would reach the level of 902 at or near station 540. Of course, as the 100 feet grade would reach the bottom at station 540, a grade of 116 feet, and, withal, upon a line lengthened by curvature would reach the level of 902 higher up the stream; so that a lighter grade should be run back from station 540 to meet it, wherever it comes down to the bottom.

In regard to curvature, it is advised to fit the line pretty closely to the ground, as it is not desirable to cut or fill deeply upon slopes so steep as those of Mount Webster or the Willey Mountain. Curves of 1000, or even 700 feet radius, will give a good line, care being taken to put in a tangent at reversions.

Concerning the grade to be adopted from the Summit *westward*, Mr. Latrobe remarks that there is a relation between that and the Saco slope which should not be overlooked. As the grade is increased on the eastern side, it may also be increased on the western side, due regard being paid to the preponderance of traffic. With a grade of 116 feet upon the eastern slope,

one of 78 feet may be employed upon the western; with 150 upon the eastern, 106 upon the western; and with a grade of 200 feet per mile on the Saco, a grade of 150 feet may be adopted on the Ammonoosuc slope. This, however, assumes a perfectly regular and reciprocal trade, in the precise proportion stated, of one to three, which would rarely obtain in practice.

The obvious reason why, as the grade is increased on the eastern side, it may also be increased on the western side, is, that the cars which are, and correctly, assumed to be the same in number and weight in both directions, irrespective of their freight, constitute a constant quantity, together with the engine and tender, which bears a larger and larger proportion, as compared with the freight, to the weight of the whole train, and consequently causes the aggregate weight to approach equality more and more nearly in each direction of movement.

These remarks apply to the grades immediately adjacent to the Summit, and not to the entire gradient system of the road, upon which, if divided into suitable stages, different classes of grades may prevail, to be worked by different classes of engines; and it may be observed that if circumstances compel the adoption of a very high grade upon the Saco slope, and if one nearly as high upon the Ammonoosuc would effect a great saving of capital, the assistant power which would in any case be employed at the Summit would be made to help the trains across it in both directions.

Guided by the preliminary location represented in Plate IV., and also by the considerations submitted by Mr. Latrobe, the final location was undertaken, and with great skill and the utmost patience pushed to a most successful completion.

Commencing at the Notch, and running down the Saco, the revised or final location descends at a less rapid rate than that shown upon the plan, keeping a position somewhat higher upon the lower slopes of the mountains than that occupied by the preliminary line, and reaching the bottom of the valley about a fourth of a mile above the old Crawford Place. From Lawrence's to the Summit, a distance of $10\frac{1}{3}$ miles, the average grade is $98\frac{3}{4}$ feet per mile, the maximum being 116 feet; and of the eight continuous

miles which embrace all of the maximum grade the average is only 107 feet per mile. The alignment is remarkably good throughout, not only in its general direction, but in its details; there being but five cases where a radius less than 955 feet has been employed, and these so short that all together they amount to less than half a mile of the line. The least radius of curvature is 637 feet, and where these curves occur the grade has been reduced to 75 feet per mile.

The favorable conditions of the location above are secured with an amount of excavation and embankment far below that encountered upon the majority of roads traversing rough mountain passes, and hardly exceeding the average of roads in many parts of New England. The grade line has been purposely kept quite high — a very essential precaution, on account of snow, and one which has been altogether too much neglected in the location of railways in the Northern States.

Tyrone and Clearfield Railway.

Upon the Tyrone and Clearfield Railway, a branch of the

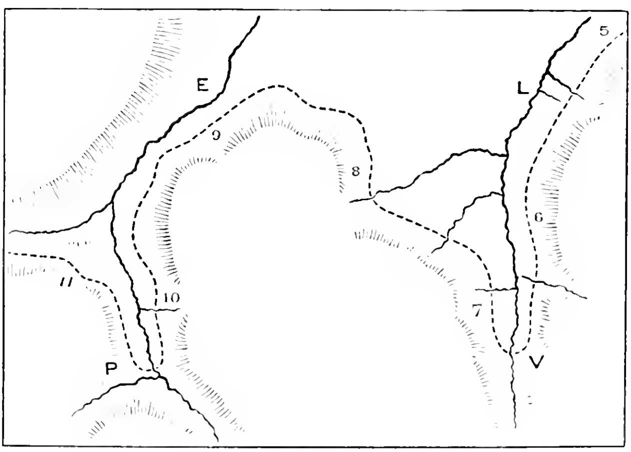


Fig. 20.

Pennsylvania Railroad, there are several very sharp curves and steep grades. At Vanscoyoc, V, in Fig. 20, there is a semicircle

of 12° curve (radius 478 feet), and at Mount Pleasant, P, in the Fig., a semicircle of from 15° to $16^{\circ}\ 40'$ curve (radius 383 to 345 feet). The rise from the "Intersection," at the junction with the Bald Eagle Valley road, 3 miles from Tyrone, to Emigh's Summit, is 1064 feet in 10 miles, or 106 feet per mile. From the 4th to the 7th mile the rise is 385.8 feet, or 128.6 feet per mile, the maximum being 138 feet. The descent westward, from the summit to Osceola, is 557 feet in 6 miles, or 92.8 feet per mile, opposed to the traffic. The heavy trains upon this road are worked in the same manner as those upon the main line of the Pennsylvania Railroad, viz., by an extra engine on the front, and in the case of long trains a third engine pushing behind.

The Brenner Railway.

The Brenner Railway, passing from Innsbruck, on the River Inn, a tributary of the Danube, to Botzen, upon the Eisack, and thence to Verona, runs through the heart of the Tyrol, and encounters greater natural difficulties than any railway in Europe.* The distance from Innsbruck to Botzen is 78½ miles. Leaving Innsbruck, which is 1899 feet above the sea, the road runs up the valley of the Sill, ascending for 22½ miles at an average rate of 114.6 feet per mile, and for the greater part of the way at the rate of 132 feet per mile. The summit at the Brenner Station is 4485 feet above the sea, being the watershed between the Black Sea and the Adriatic. From the summit, the road follows the Eisack, 56 miles, to Botzen, 860 feet above the sea; the steepest grade being 118.8 feet per mile, this being nearly continuous for the first 14 miles from the summit. A short distance below the village of Gries the road ascends the Schmirnthal (a lateral valley), doubling around for development, and tunnelling under the promontory between the Schmirnthal and the Valserthal, as shown in Fig. 21. Again, at the Pflersch-

* Not excepting the line from Vienna to Gratz, over the Soemmering, or that from Bologna to Florence, across the Apennines.

thal (a lateral tributary to the Eisack), at a short distance above Sterzing, the development shown in Fig. 22 is made, the line

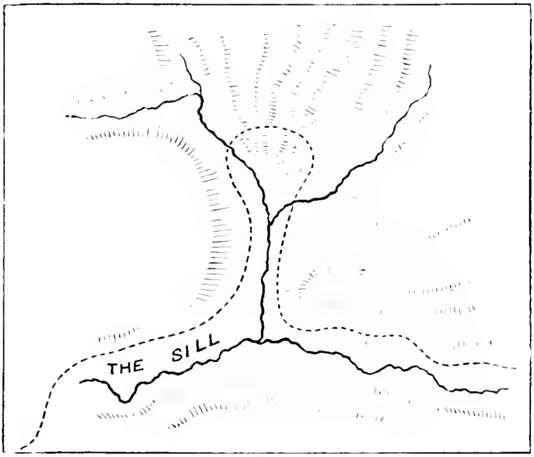


Fig. 21.

descending rapidly, but being confined to one slope only, instead of crossing the valley as in Fig. 21.

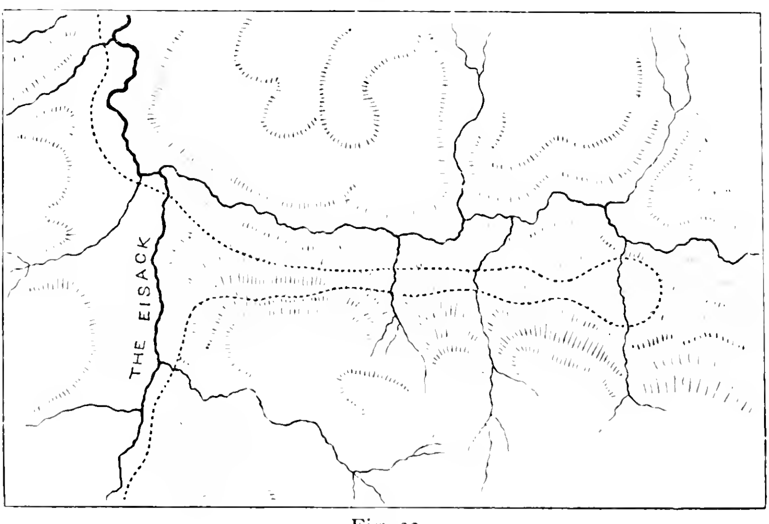


Fig. 22.

Additional Examples of Steep Grades and Sharp Curves.

There are two points upon the railroad from Bilbao to Miranda, in the north of Spain, at the entrance of the basin of Ordima, distant only 600 yards, measuring across the neck of the basin, which are $8\frac{1}{2}$ miles distant along the line of the road, and which differ 456 feet in elevation.

Besides the 93 feet grades of the Pennsylvania Railroad, the 105 of the Pacific, the 116 of the Baltimore and Ohio, the 132 of the Brenner, and the same upon the Soemmering, may be noted six miles of 146.6 feet upon the line from Turin to Genoa, grades from 133 to 145 feet upon the Lehigh Valley Railroad, of 143 feet in India, of 195 in the Mauritius, long reaches of from 160 to 211 feet in South America, and a short grade upon the Jeffersonville, Madison, and Indianapolis Railroad, which rises 394 feet in a mile and a quarter, or 320 feet per mile. All of the above are for permanent use. For temporary purposes may be mentioned the "Mountain Top Track," constructed by Charles Ellet, at the Rock Fish Gap crossing of the Blue Ridge, upon the Virginia Central Railroad. The total length of this track from the eastern base to the summit was 12,500 feet, or 2.37 miles, and the rise 610 feet; the western slope was 10,650 feet long, or 2.02 miles, and descended 450 feet. The maximum grade on tangents was $5\frac{6}{10}$ feet per 100, or $295\frac{68}{100}$ feet per mile; the general grade on tangents being $5\frac{3}{10}$ feet per 100, or $279\frac{84}{100}$ feet per mile. The least radius of curvature was intended to be 300 feet; but at one point this was reduced to 275 feet, in order to save work. The usual grade on these abrupt curves was $4\frac{5}{10}$ feet per 100, or $237\frac{6}{10}$ feet per mile. It was supposed by Mr. Ellet, in establishing the plan of this road, that a difference of 58 feet per mile between the grade adopted on tangents and that on curves of 300 feet radius, would be an adequate compensation for the increased resistance due to curvature. The result, however, proved that the engines labored much more in drawing the

trains through the curves than upon the steeper ascents of the straight lines.*

Temporary tracks upon the Chesapeake and Ohio Railroad have been worked with safety and economy for many years, the maximum grade being 308 feet per mile, and this on curves of 500 feet radius. During the construction of the Baltimore and Ohio road, temporary lines were worked over mountains, beneath which tunnels were in process of construction, having grades of 530 feet per mile; and over these grades a mixed traffic was safely and regularly worked for a long time.

* The engines used upon the above grades were six-wheeled connected tank engines, by Baldwin, weighing 50.000 pounds, or, with wood and water, 55.000. Drivers, 42 inches; outside cylinders, $16\frac{1}{2} \times 20$; wheel-base, 9 feet 4 inches. The common load was three eight-wheeled freight cars, loaded, weighing each, with its load, about 14 tons; making a total seldom less than 40 tons, and never over 50 tons. The speed adopted was from six to seven miles an hour, the engines making four trips of 8 miles each per diem, during a period of three years. The engines employed upon the steep grades referred to upon the other railways mentioned above, will be described in a following chapter.

CHAPTER IV.

LAYING OUT WORK.

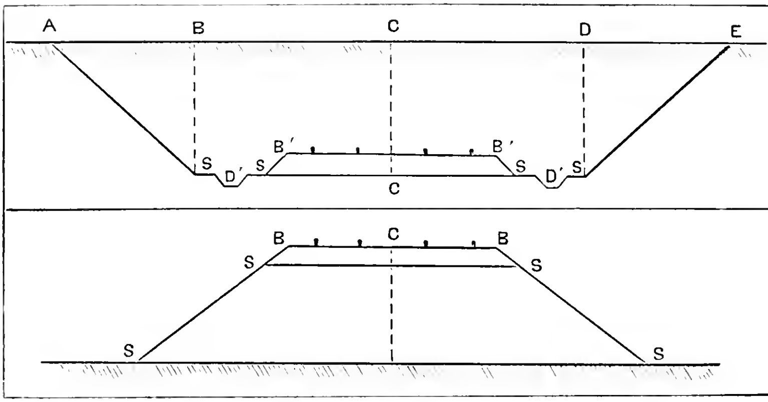
THE running of the line consists in placing a stake at each 100 feet upon the straight portions (tangents) and upon curves of large radius, and at each 50 feet when the curvature is sharp. At the tangent points of curves, and at points of compound and reversed curvature, a larger and more permanent stake or post should be placed in the ground, the exact point being fixed by means of a nail driven into the top. Lest these stakes should be disturbed in the process of construction of the works, their exact distance from several points outside of the ground to be occupied by the road should be carefully measured, and entered in the engineer's note-book, that they may at any time be replaced. The stakes above referred to show the position of the centre line of the railway, and form the base line from which all operations of construction are carried on.

THE SIDE SLOPES.

The next step in preparing for the excavation, is the placing of the side stakes, which are at a distance from the centre equal to half the width of the road-bed. Following the above comes the putting down of the slope stakes, A and E, Fig. 23, which are placed upon each side of the centre, at the distance to which the slope commencing at the outer edge of the side ditch will extend, depending upon the angle of the slope, the depth of the cutting, and the width of the road-bed. The simplest case that occurs of obtaining this distance is when the natural surface is horizontal.

Let A B C D E (Fig. 23) represent this surface, A S and E S the side slopes, D′ D′ the ditches, and B′ B′ the ballast; then, if the width of road-bed, B D, is 20 feet, the depth of cutting 10 feet, and the slopes $1\frac{1}{2}$ horizontal to 1 vertical, A B will be once and a half B S, or 15 feet, and B C being 10 feet, the required distance, A C or C E, will be 25 feet.

In the case of an embankment, we have the section shown in



Figs 23 and 24.

Fig 24, the road-bed being narrower than in the cutting, as there are no side ditches. The horizontal distance, therefore, from the centre line out to the foot of the embankment will be less than the distance from the centre line to the side slope in excavation, even when the depth of the filling is the same as the depth of cutting.

When the natural surface of the ground is inclined, the setting of the slope stakes is less simple, and one or more trials are needed before the exact point is found. Thus, in Fig. 25, A D B being the natural surface, we see at once that the up-hill slope stake is at a greater, and the down-hill stake at a less horizontal distance than if the ground was level. In this case we assume the point B, find with the level its height above the road-bed H K, multiply this height by the slope, add half the width of road-

bed, and find how nearly the distance thus calculated agrees with the assumed distance C B. If within a few inches it will answer; if not, a second trial will fix the place.

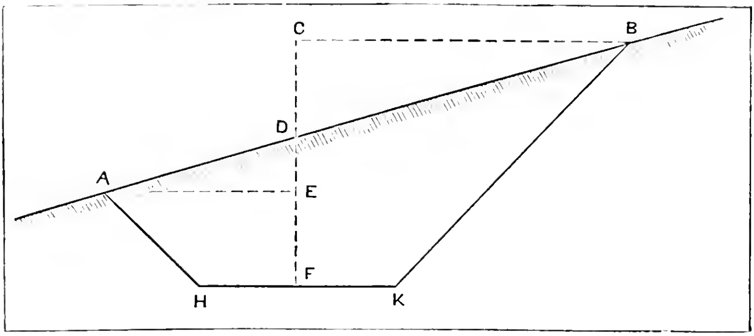


Fig. 25.

Suppose we wish to find the horizontal distances D E and H K, Fig. 26, at which to put in the stakes for the bottom of the slopes of an embankment. the road-bed being 15 feet wide, the

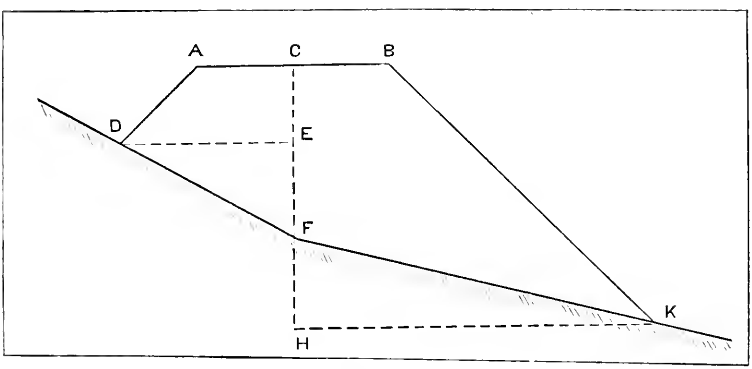


Fig. 26.

slopes $1\frac{1}{2}$ to 1, and the filling at the centre, C F, 13 feet. Assume D as the point for the upper stake: let its height above F, as found by the level, be 8 feet; C E is then 13—8, or 5 feet, and

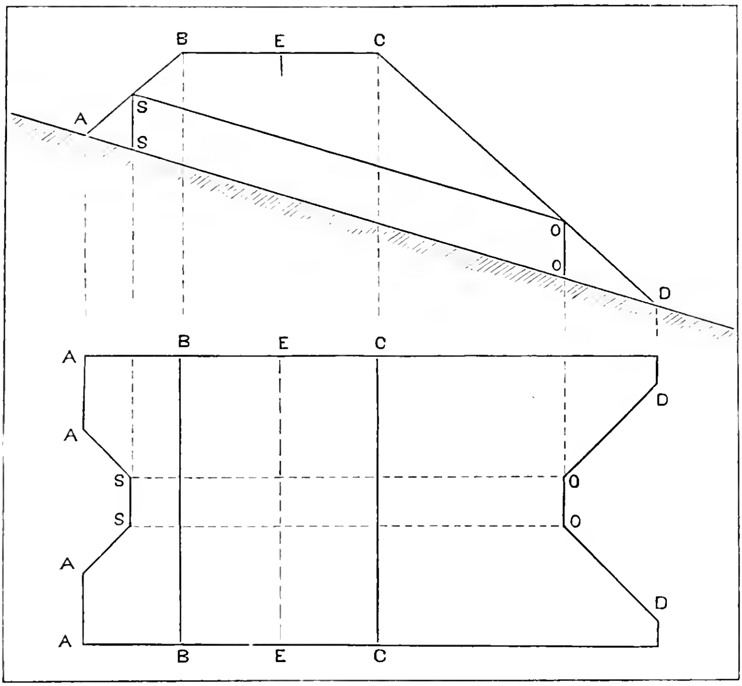
D E is A C + $1\frac{1}{2}$ C E, or $7\frac{1}{2}$ + $7\frac{1}{2}$, or 15 feet. If the horizontal distance from F to D, or E D, measures 15 feet, the assumed point, D, is correct; if not, a second trial must be made. For the lower side we assume the point K, add F H to F C, and once and a half C H plus C B must give H K.

Laying out Culverts.

The length of any structure, such as a culvert, passing under an embankment, is less than the distance between the slope stakes, by twice the height of the structure multiplied by the inclination of the slope. If a culvert 6 feet high passes under a bank 30 feet deep and 15 feet wide on top, the slopes being $1\frac{1}{2}$ to 1, the length of the culvert will be $15 + (2 \times 30 \times 1\frac{1}{2}) - 6 \times 1\frac{1}{2} \times 2$, or $105 - 18$, or 87 feet. The length of an oblique structure is of course greater than that of one at right angles to the road, the length depending upon the obliquity.

When the natural surface is horizontal, the length of any structure passing under an embankment will lie half on each side of the centre line. When the natural surface is inclined, the ends of the structure will be at different distances from the centre line, according to the slopes of the ground. This is seen in Figs. 27 and 28, the first of which represents the section of an embankment, and the second the plan of a portion of the same. The lines S S and O O, representing the ends of a culvert passing beneath the embankment, are seen to be at different distances from the centre line. We may find the position of the points S and O upon the ground, by first fixing in the manner already described the points A and D, and measuring *in* from these points the distances A S and D O, depending upon the slopes A B and A D. In the case of the upper end, the distance of S S from A will be less than if the natural surface was level; at the lower end, the distance from D to O will be greater. Having found the distances of S S and O O from the centre line, we get the position and length of the wing walls of the culvert by drawing a line from S,

at any desired angle, to intersect the slope A A; and upon the lower side of the embankment we get in the same manner the lines O D, O D, the latter being of course longer than the wings upon the upper side, A S, A S. In practice, the various distances above referred to are easily found by plotting the cross slope of the ground and the section of the embankment upon a scale depending upon the required accuracy of the work.



Figs. 27 and 28.

Laying out Bridge Work.

In laying out the abutments for bridges, there are numerous cases to be considered; as, whether the bridge is right or oblique, upon a level or a grade, upon a curve or a straight line, and whether

the natural surface is horizontal or inclined; and, according as these elements are more or less combined, is the operation more or less difficult. The position and form of abutments and wing walls depend so much upon the various conditions affecting each particular case, that any attempt to lay down general rules for such work would be of little use.

In curving a viaduct, consisting of a series of arches which exert a thrust upon the masonry, the piers should be made radial to the centre of the located curve, and the springing lines should be made parallel to the axes of the arches. The pier thus becomes a wedge, and should be buttressed upon the outside of the curve, in order to resist the resultant of the thrust of the two adjoining arches.

General Remarks upon Laying out Work.

In placing the stakes for any structure, we should have them so far outside of the work that they will remain undisturbed while operations are going on. The pegs for cutting pits and trenches for foundations may be placed at the angles of the latter; but the working points must be so placed that lines stretched from one to the other will define the masonry.

Two stakes, at a sufficient distance apart upon the land, will fix any *line* upon the water; and two sets of stakes, upon different lines upon the shore, will, by their intersection, fix any *point* upon the water with accuracy sufficient for many purposes. For exact work, however, a Transit or Theodolite should be employed to fix a *line*; and two angular instruments, in well chosen positions, will determine any *point*. A permanent bench, or reference mark for levels, should be established with care in the immediate neighborhood of any proposed structure, from which the elevations of the various parts may be obtained. Such bench marks should also be fixed at the commencement of long cuttings, and generally at intervals of from 500 to 1000 feet along the works, a list of such elevations being entered in the engineer's note-book.

Laying out Tunnels.

The establishment of a correct centre line for a tunnel is a very important operation, and one requiring the utmost care. The fixing the line at the bottom of the shafts demands every precaution, owing to the short distance between the only points that can be transferred from the surface to the bottom. The centre line of the road is first run over the elevation to be tunnelled, and carefully fixed by permanent monuments placed upon prominent points. On account of gases and heated vapors, which rise from the shafts, and mix irregularly with the outside air, there is always more or less atmospheric disturbance, and consequent variation in the position of the centre line as determined by the transit at different times. It is thus necessary to fix the line by repeated trials under different atmospheric conditions, to note the variations, and finally to correct the adjustment by averaging the varying results.

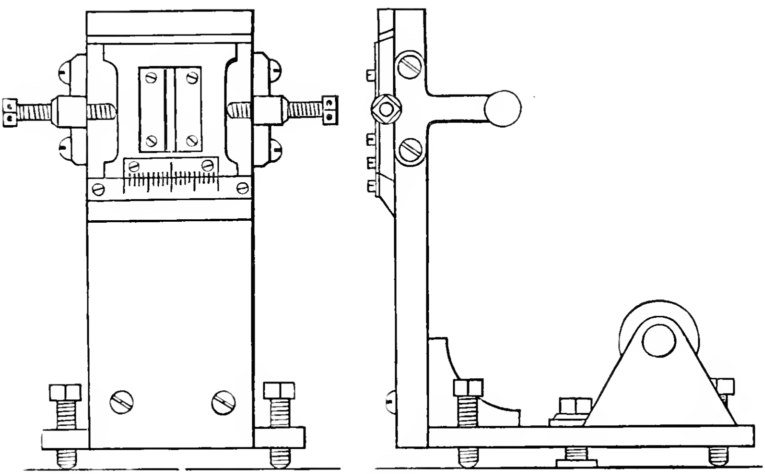


Fig. 29.

The method followed at the Hoosac Tunnel, with remarkable success, was as follows: Upon each side of the shaft, and on the line of the road, an instrument such as shown in Fig. 29 was firmly bolted

to a solid foundation. This instrument consists of a small plate with a vertical slit so arranged as to move towards either side over a horizontal scale, the precise position being shown by a vernier attached to the plate. The transit being placed correctly in position, the centres of the vertical slits, or the zeros of the verniers, are sighted accurately into line. This operation is repeated a sufficient number of times, and from the successive readings a mean position is computed, and the slides fastened. Two fine steel wires, resting close against the opposite sides of the slits, are tightly stretched from plate to plate, thus forming two parallel lines across the shaft; and through the space between these lines pass vertical copper wires, holding heavy brass plummets hanging in buckets of water at the bottom of the shaft. These copper plumb-lines are enclosed in wooden tubes, to protect them from dropping water and from currents of air. A floor is thrown across the shaft near the bottom and a few feet above the roof of the tunnel. Upon this floor horizontal scales are placed, directly in front of which the vertical wires oscillate. A series of observations fix the mean position of the wires upon the scales, and by means of the points thus determined the centre line of the tunnel is produced in either direction.

The accuracy attainable by the above method is well shown by the results at the Hoosac Tunnel. The heading from the central shaft ran eastward 1563 feet, and met the heading from the eastern end with an error of but *five sixteenths of an inch.* The heading from the central shaft towards the west ran 2056 feet, and met the heading from the western end with an error of but *nine sixteenths of an inch.* By a final test of the line, carried down through the central shaft 1028 feet deep, the error in a length of 23 feet was found to be only *one four-hundredth of an inch;* and at the meetings of the eastern headings less than one fifth of an inch in 1563 feet. The error in levels east of the shaft was $1\frac{1}{4}$ inches, and west of the shaft $1\frac{1}{2}$ inches only. The apparatus shown above was designed by Mr. Carl O. Wederkinch, assistant engineer in charge of the central section of the Hoosac Tunnel,

under whose direction the remarkable results above noted were obtained.

When a tunnel is upon a curve, the line may be laid off from a tangent, established as above. Levels may be transferred from the top to the bottom of a shaft by a series of wires fastened together like a surveyor's chain, or by a single wire — the length having been carefully obtained, and also tested after use, to allow for stretching.

In laying out the Mont Cenis Tunnel, the length, 7.5 miles, was determined by an elaborate trigonometrical survey. An observatory was established upon the highest summit, more than a mile vertically above the portals, from which could be seen a secondary observatory in each direction, placed at some distance from, and opposite to, each entrance to the tunnel. These observatories were built of stone, and every care was taken to insure the perfect stability of the instruments used. Such arrangements were very necessary, as the tunnel had no shafts, and the lines ran in nearly four miles from each end before meeting.

CHAPTER V.

EARTHWORK.

Excavation and Embankment. — The Profile.

The quantity of excavation and embankment, expressed in cubic yards, is required to be known in order to compare the amounts of work to be done upon different trial lines which may have been surveyed, or in order to estimate the actual cost of the route finally selected. The survey has of course included the making of a complete profile of the road, and such cross levellings at right angles to the line as may be needed to show the form of the surface. The profile is commonly plotted upon long rolls of paper, engraved in red, with the system of horizontal and vertical lines shown in Fig. 30. The vertical lines are a fourth of an inch apart, each tenth line being ruled heavier than the intermediate ones. Horizontally, the sheet is first divided by the heavier lines, about eight tenths of an inch apart: this distance is divided into five equal parts, by lighter lines, and these smaller divisions are subdivided by still lighter lines into five parts, each of the smallest spaces being about one thirtieth of an inch. In common practice each of the smaller horizontal divisions represents 100 feet, and thus the distance between the heavy vertical lines is 1000 feet. The smallest divisions upon the vertical scale are taken as one foot. The vertical scale is thus about 13 times larger than the horizontal: a disproportion which enables small heights to be appreciated, without making the profile inconveniently long.

At each 100 feet upon the ground a stake is placed in the located line, and at each "station" thus fixed the elevation is measured with the level. Suppose that our notes give us the following heights, above any datum or base line that may have been assumed: —

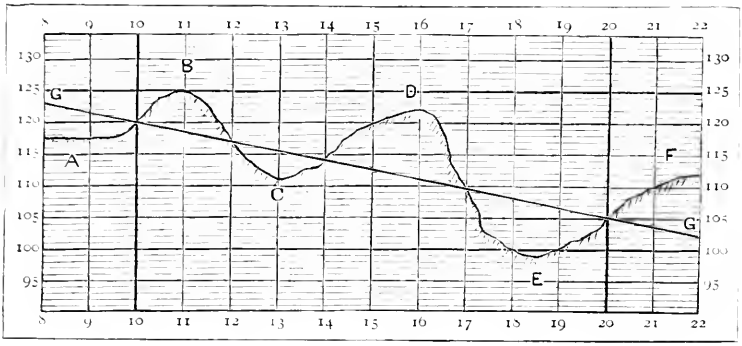


Fig. 30.

Station.	Elevation.	Station.	Elevation.	Station.	Elevation.
9 . .	117.5	14 . .	114.0	19 . .	100.0
10 . .	120.0	15 . .	120.0	20 . .	105.0
11 . .	125.0	16 . .	122.0	21 . .	110.0
12 . .	117.0	17 . .	110.0	22 . .	112.0
13 . .	111.0	18 . .	100.0	23 . .	108.0

At the intersection of the vertical line 9, and the horizontal 117.5, we make a point; at the intersection of 10 and 120 another; at the intersection of 11 and 125 another; at 12 and 117 another, and so on: and through the points thus fixed we draw the surface line as in the figure. Next, we establish the grade line of the road, ascending, descending, or level, according to the general form of the profile, due regard being had to balancing the amounts of excavation and embankment, and to the effect of the inclines upon the traffic.* Suppose that by so doing we have the grade shown by the inclined line in the figure. We get by inspection, with sufficient accuracy for an approximate estimate, the depth of cutting or filling at each station.

* This may be done by stretching a piece of fine black silk along the length of the profile, and moving it up or down, and inclining it according to the undulations of the ground. The thread is better than a ruler, as we can see both above and below it, and thus find at once the effect of a change in the grade line, both upon the excavations and embankments.

OF THE CUBIC AMOUNTS.

Knowing the depth of cutting, the width of the road-bed intended, and the angle of the side slopes, we obtain the area of the section across the line of the road at any point ; and having the areas at any two stations, and the distance from one to the other, we get the cubic amount of earth to be moved between those stations. If the natural surface was in every case horizontal, transversely, the areas would be simply the mean width of the top and bottom, multiplied by the height. The surface, however, takes every variety of form, and hence an equal variety of problems for obtaining the areas. The making of estimates is very much facilitated by tables, which are easily prepared, giving at sight the cubic contents of any prism when the depth of the

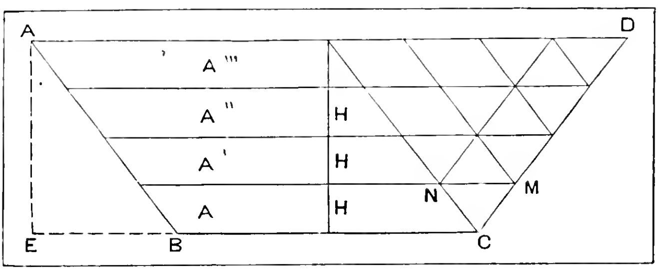


Fig. 31.

cutting at each end is known. Let B C, Fig. 31, represent the width of the road-bed, A E the depth of cutting, and A B and C D the side slopes. If we call B C, B ; the horizontal member E B of the slope R, and the height A E, H ; the area is —

$$(B + R) \times H = A.$$

If the distance from station to station is taken at 100 feet, as it generally is, and the amount is required in cubic yards, the above formula becomes

$$\frac{(B + R) \times H}{0.27} = C.$$

An examination of Fig. 31 shows that the area of A' exceeds that of A by twice the area of the small triangle C N M; and that A'' exceeds A' by the same amount. This increase being constant, we have only to find the area of A, and for the area of $A + A'$ add the increase to twice A; whence the rule: Having found the increase, for the second area add the increase to twice the first; for the third add three times the increase to three times the first; and for the fourth area add six times the increase to four times the first area. The areas in the figure above are thus: —

A	A
$A + A'$	$2A + I$
$A + A' + A''$	$3A + 3I$
$A + A' + A'' + A'''$	$4A + 6I$

In forming tables for the computation of earthwork, we must know the width B C, or the width of the road-bed, the angle of the slopes, and the partial heights H, H, H, which latter may be taken at a foot, or a tenth of a foot, according to the accuracy required. In preparing such a table, we may, if we prefer, work at once upon the cubic amounts instead of the areas, by cubing the first area and the increase mentioned above. A table thus prepared may be of the following form: —

Depth in Feet.	Road-bed for Single Track, 18 Feet wide, and Slopes.			Road-bed for Double Track, 28 Feet wide, and Slopes.		
	$1\frac{1}{4}$ to 1. *Earth.*	$1\frac{1}{2}$ to 1. *Earth.*	$\frac{1}{4}$ to 1. *Rock.*	$1\frac{1}{4}$ to 1. *Earth.*	$1\frac{1}{2}$ to 1. *Earth.*	$\frac{1}{4}$ to 1. *Rock.*
1	71	72	68	108	109	105
2	152	155	137	226	230	211
3	242	250	208	353	361	319
4	341	355	281	489	504	430
5	449	472	356	634	657	542
6	567	600	433	789	822	656
7	693	739	512	953	998	771
8	830	889	593	1126	1185	889
9	975	1050	675	1308	1383	1007
10	1130	1222	759	1500	1592	1130

DIFFERENT METHODS OF OBTAINING THE QUANTITIES.

The amounts upon any profile may be calculated by averaging the areas at the ends of each 100 feet length; or by the area based upon the mean height of the two ends, termed the middle area; or by the Prismoidal Formula, which is as follows: *To the sum of the end areas add four times the middle area. Multiply the sum by one sixth of the length.* The result by the first mode will be a little too large, by the second mode a little too small, while the Prismoidal Formula gives the correct amount.

By means of the table, we work at once upon the cubic quantities, instead of upon the areas.* Suppose we have a section 100 feet long, 6 feet deep at one end, and 10 feet at the other, the road-bed being 18 feet wide and the slopes $1\frac{1}{2}$ to 1. By averaging the end areas we have, opposite the depth 6, and under the middle column for the 18 feet road-bed, 600 yards; opposite the depth 10, we have 1222 yards; the mean of the two is 911 yards. By the second method, the mean height, or depth, being 8 feet, the amount is 889 yards. By the Prismoidal Formula, we have —

Depth 6, ...	600 yards.
Depth 10, ...	1222 yards.
Depth 8 (889) × 4, ...	3556 yards.
The sum, ...	5378
One sixth, ...	896 yards.

The method of middle areas, for the rough estimates needed in the comparison of different surveyed lines, is very convenient, and enables us to work rapidly; but, as before remarked, it furnishes results a little too small. The amount of the deficit depends more upon the difference of the cutting at the two ends,

* Being provided with a profile, upon which the grade line is drawn, and a table like the one above, let one person read off the cuttings, from inspection, at each 100 feet, a second person give the corresponding number of yards in the table, and a third person set the amounts down, care being taken to separate the excavations from the embankments.

than upon the actual depth. Thus, with a cutting of 10 feet at one end, and 30 feet at the other, the difference in the quantities furnished by the method of middle areas and by the Prismoidal Formula is 185 yards, or about 5 per cent.; while with a depth of 15 feet, at one end, and 25 at the other, the mean depth being the same as before, the difference by the two methods is 46 yards, or only 1.28 per cent. The percentage of error decreases, with the same difference between the ends, as the actual depth becomes greater. Thus with the ends at 30 and 40 feet, the quantities furnished by the two methods differ only $\frac{1}{2}$ of a per cent.

In making estimates, we may use the same tables for embankment as for excavation. Indeed, for work intended to be only approximate, although the embankment is narrower than the cutting by the width of the ditches, since the material when put into the bank shrinks or compresses from $\frac{1}{12}$th to $\frac{1}{8}$th, as hereafter described, we may even use the same width of road-bed, and may establish the grade line so as to make the cuttings and the fillings balance.

In the above remarks the natural surface transverse to the line of the road has been supposed to be horizontal; or the work has been, as it is termed, in level cutting. When the surface is inclined, or is irregular, the area is less easily obtained. When the ground is considerably broken, small transverse profiles, termed cross sections, are made, being plotted upon paper ruled for the purpose in fine squares, the vertical and horizontal divisions being alike. These squares being taken as one foot each, we get easily the area of any section, however rough the surface of the ground may be.

Borrowing and Wasting.

In the execution of earthwork, it is not always advisable to make the whole of an embankment from the adjoining cuttings. The length of the haul may thus be rendered too long. In such cases it is customary to waste a part of the cut, depositing the earth in the most convenient place, and to borrow material from

some nearer point for the embankment. A great deal has been written upon the problem of the most economical mode of obtaining and transporting earth, and numerous complex formulæ have been furnished for the guidance of engineers. Local circumstances, however, more than anything else determine the points from which earth shall be borrowed, and the routes over which it shall be moved.

To find the cost of the movement of the earth upon any section of a road, we must know the character of the earth, the total amount to be moved, and the average haul; the latter being the distance through which, if the whole amount of excavation upon a given length was transported, the product of that distance by the total amount of earth would be the same as the sum of the products of the partial amounts by their respective distances. To find this average haul, we first ascertain the distance, by inspection of the profile, between the centres of gravity of each mass before and after its removal; and next, we divide the sum of the products of the partial amounts by their respective hauls, by the total amount. Thus, let column 1, in the table below, represent the partial amounts, in cubic yards; column 2, the respective hauls in feet; and column 3, the product of each amount by its haul.

$$\begin{array}{rcrcr}
1000 & \times & 200 & = & 200{,}000. \\
2000 & \times & 300 & = & 600{,}000. \\
5000 & \times & 400 & = & 2{,}000{,}000. \\
8000 & \times & 600 & = & 4{,}800{,}000. \\
\hline
16{,}000 & & & & 7{,}600{,}000
\end{array}$$

Then, 7,600,000, divided by 16,000, or 475 feet, is the average haul; or, generally, if m m′ m″ m‴ represent the partial amounts, and d d′ d″ d‴ the corresponding distances, S being the total amount, and D the average haul, we have

$$\frac{m\,d + m'\,d' + m''\,d'' + m'''\,d'''}{S} = D.$$

Cost of Executing Earthwork.

The movement of earth is effected by shovels, barrows, carts, or cars. The elements entering into the problem of the most economical mode of transport are numerous and variable. The determination of the kind of vehicle best adapted to any particular distance, the relation between the number of picks and shovels, and between the excavators and the transporters for different kinds of earth, and for different distances, has been undertaken by many engineers. Among the best papers upon the subject are those of Ellwood Morris, in the Journal of the Franklin Institute,* and the articles by J. C. Trautwine, C. E., in the latter part of his excellent work upon Excavation and Embankment.†

The several items that go to make up the total cost of earthwork, are the loosening of the earth, either by picks or ploughs, the putting it into the barrow or cart, the moving and emptying it, the spreading it out upon the embankment, the return of the vehicle, the keeping the road in proper order, the wear and tear of tools and wagons or barrows, the interest on the cost of the equipment, and finally the wages of the overseer's and the contractor's profit. The loosening of the material in shallow cuttings and in light soils is done best by the plough. In deep cuttings, the earth being undermined at the ends, falls of itself. For short distances, 10 to 20 feet, the earth, if loose and dry, may be moved by shovels ; from 20 to 200 feet, barrows may be employed, running upon a plank ; for over 200 feet, carts will be found more economical ; and for hauls over 500 feet, where a large amount of work is to be done, a track, with cars drawn by horses, will be found profitable. These distances, however, depend much upon the difficulty of getting out the earth. With hard clay, requiring two picks to a shovel, and with a small surface to work upon, two

* Vol. II., 3d series, p. 164. 1841.

† A New Method of Calculating the Cubic Contents of Excavations and Embankments, by the Aid of Diagrams: together with Directions for Estimating the Cost of Earthwork. By John C. Trautwine, C. E.

carts upon an ordinary road will take away all that a dozen men can get out; while with an easy soil, where one pick will keep half a dozen shovels busy, a larger number of vehicles will be required, or a quicker haul, which may be got by putting down a track. The less the haul, or the greater the speed of transport, the fewer may be the number of carts to take away a given amount of material. The chief point to be gained is to arrange the different classes of laborers so that none shall be kept waiting. Everything depends upon the tact for management possessed by the overseer.

Mr. Morris found, upon the Chesapeake and Ohio Canal, that 28,100 cubic yards of embankment, made of loam, with an average haul upon a level of 650 feet, cost $20\frac{1}{10}$ cents per yard; the day being 10 hours, the wages of a laborer 106 cents, and of a cart 157 cents. Another embankment of 42,140 yards, of sandy earth, with an average haul of 900 feet upon a level, cost $21\frac{8}{10}$ cents per yard; the day being 10 hours, the wages of a laborer 110 cents, and of a cart 160 cents. Another embankment of 23,500 yards, of sandy earth, with an average haul of 600 feet slightly ascending, cost 22 cents per yard; the day being 10 hours, the wages of a laborer 120, and of a cart 165 cents. Another embankment of 36,000 yards, of sandy earth, with an average haul of 920 feet upon a level, cost $22\frac{6}{10}$ cents per yard; the day being 10 hours, the wages of a laborer 118 cents, and of a cart $162\frac{1}{2}$ cents. Another embankment of 22,075 yards, of loam, with an average haul of 1000 feet upon a level, cost $27\frac{9}{10}$ cents per yard; the day being 10 hours, the wages of a laborer 125, and of a cart 175 cents. Tabulated, the above data stand as below.

Amount of Embankment in Cubic Yards.	Kind of Earth.	Average Haul in Feet.	Wages of Laborer, in Cents	Wages of Cart, in Cents.	Cost per Cubic Yard in Cents.
21,800	Loam.	650	106	157	20.1
42,140	Sandy earth.	900	110	160	21.8
23,500	Sandy earth.	600	120	165	22.0
36,000	Sandy earth.	920	118	$162\frac{1}{2}$	22.6
22,075	Loam.	1000	125	175	27.9

The following figures, based upon Mr. Trautwine's table, show the total cost, to the nearest cent per cubic yard, exclusive of contractor's profit, of different kinds of earth and of rock taken out and hauled by carts to different distances. All such figures are necessarily only approximate, being obtained from data which, though assumed as permanent, are in reality quite variable : —

Haul in Feet.	Sand. Wages being in Cents.			Loam. Wages being in Cents.			Heavy Earth. Wages being in Cents.			Stiff Clay. Wages being in Cents.			Rock. Wages being in Cents.		
	100	125	150	100	125	150	100	125	150	100	125	150	100	125	150
100	12	15	18	14	17	21	17	21	25	20	25	30	60	75	90
500	15	18	22	17	21	25	19	24	28	22	27	33	65	81	97
1000	19	23	28	21	26	31	23	29	34	26	32	39	72	90	108
1500	22	27	33	24	30	36	27	33	40	30	37	45	78	97	117
2000	26	32	39	28	35	42	30	37	45	33	41	49	84	105	126
2500	29	36	43	32	40	48	34	42	51	37	46	55	90	112	135
3000	33	41	49	35	43	52	38	47	57	41	51	61	96	120	144
3500	37	46	55	39	48	58	41	51	61	44	55	66	103	129	154
4000	40	50	60	42	52	63	45	56	67	48	60	72	109	136	163

Shrinkage of Earthwork.

Most earths, when newly broken up, occupy more space than when in the natural state; but when placed in embankment they occupy considerably less. Thus, if in arranging our cuttings and fillings we make the amounts upon the profile to balance, we shall find a lack of material before the banks are completed. Railway embankments, especially when rapidly made, are found to settle, even for years, requiring a continual addition of material to keep them up to the grade line.

Mr. Morris furnishes the following facts: An embankment of 6262 cubic yards required 6970 yards of excavation; the shrinkage or compression in the bank thus being 708 yards; the material being a yellow, clayey soil. An embankment of 23,571 cubic yards required 25,975 yards of excavation; the compression being 2404 yards; and the material the same as above. An embankment of 9317 cubic yards required 10,701 yards of excavation; the compression being 1384 yards, and the material a

light sandy soil. The whole 39,150 cubic yards of embankment, above, thus required 43,646 yards of excavation; the difference being 4496 yards, or just about *one tenth*. Mr. Morris concludes that the rates of compression in earth embankments are,—

In light sandy earths, one eighth of the volume in excavation.

Yellow clayey earths, one tenth of the volume in excavation.

In gravelly earths, one twelfth of the volume in excavation.

Rock, on the other hand, increases in volume by being broken up, and does not compress again into less than its original bulk. The same authority above quoted states that 22,625 cubic yards of hard sandstone, quarried in large fragments, made 32,395 yards of embankment; the increase being 9770 yards, or about five twelfths of the volume in the cut. Again, 16,982 cubic yards of blue slate, broken into small pieces, made 27,131 yards of embankment; the increase being 10,149 yards, or nearly six tenths of the bulk measured in the cut.

In arranging to obtain a given amount of material for embankments, we may, therefore, make the following allowances:—

Gravel will shrink eight per cent.

Gravel and sand nine per cent.

Clay and clay earths ten per cent.

Loam and light sandy earths twelve per cent.

Rock will *expand* fifty per cent.

To illustrate the need of allowing for the shrinkage of material, suppose we wish to form an embankment of 100,000 yards, and that the regular cuttings will furnish only 80,000 yards, measured in excavation; we have then to get 20,000 yards outside of the line of the road. If the cost of the earth obtained from the regular cuttings is 25 cents a yard, and that obtained from other sources is 30 cents a yard, and we neglect the shrinkage, the cost would be,—

80,000 yards at 25 cents, or	$20,000
20,000 yards at 30 cents, or	6,000
And the total	$26,000

But the 80,000 yards of excavation at a shrinkage of one tenth, will only make 72,000 yards; there is thus 28,000 yards to be obtained outside of the regular cuttings; and this amount again is to be increased for shrinkage, and becomes 31,111; and the cost is thus: —

80,000 yards, at 25 cents, or	$20,000
31,111 yards, at 30 cents, or	9,333
And the total	29,333

The estimated cost, therefore, neglecting the shrinkage, is too small by $3,333.

Cross Sections for Borrow Pits.

In obtaining earth outside of the line of the road, or, as it is termed, by borrowing, or from borrow pits, in order to know how much earth is taken, the ground should be divided into squares, of greater or less size, according to the irregularity of the surface, and the height at the corners of each square, above some common datum, should be ascertained by the level, and reference points fixed so that at any future time, by means of a new levelling, the difference between the original and the altered surface may be known, and thus the amount of material removed.

Allowance for Subsidence.

When embankments are carried up in layers a few feet thick only, such layers being carted over, the subsequent settling is quite small; but when the whole depth is carried up at once, the filling should be made somewhat higher than the intended grade line, and the full width should be maintained at the increased height, so that when the final subsidence has taken place the proper width on top may be had. The amount of the above allowance will depend much upon the quality of the earth, and

also upon the time in which the work is done. It has in some cases been specified that the embankments, when completed by the contractor, should be finished to the full width from three inches above the intended height, upon a bank five feet high, to nine inches upon a forty feet bank; intermediate heights being in proportion.

Angle of Slopes.

The transverse slopes of cuttings and embankments depend upon the nature of the soil in which the work is carried on. Gravel will stand at a slope of $1\frac{1}{2}$ horizontal to 1 vertical, and in some cases at $1\frac{1}{4}$ to 1. Clay, though remaining at a high angle when first cut, finally assumes a very flat slope, even as low as 4 to 1. When a bed of clay underlies a lighter earth, it is sometimes economical to support the clay by a retaining wall, and to slope only the overlying material. The manner in which slips occur upon high slopes of clay and clayey earths, suggests the proper form to be given to the cross section of the cutting in such cases. This is not a straight line, but a curve resembling the parabola, flatter towards the bottom of the slope where the pressure is greatest, and steeper above. Care should in all cases be taken to secure good drainage, and to protect the slopes of earthwork. When the ground is inclined transversely, a surface drain along the top of the upper slope in cuttings will prevent the water from finding its way into the excavation; and in the case of an embankment upon a similar surface, a drain parallel with the foot of the slope, upon the upper side, and a few feet distant, will prevent the water from working under the bank. Such drains should be led to the watercourse which occupies the lowest part of the depression crossed by the embankment. The slopes of both cuttings and fillings may often be protected by small open drains, passing down the slope in a diagonal direction. The side drains in long excavations should be slightly inclined, in order to insure the running off of the water.

NOTE. — Information upon the calculation of amounts, and upon the cost of executing earthwork, will be found in the work of Mr. Trautwine, already mentioned. For the theory of the stability and pressure of earth, and for remarks upon the carrying on of the various operations connected with the making of excavations and embankments, Professor Rankine's Manual of Civil Engineering (Part 2, Chap. II) may be consulted. For valuable memoranda concerning the earthwork and drainage of English and French railways, and for suggestions for the improvement of American roads, the reader is referred to the work of Messrs Colburn and Holley upon the Permanent Way of European Railways. Reference to various points relating to the conducting of the construction operations upon railways, will be found in the Appendix, Art. *General Specification.*

CHAPTER VI.

ROCKWORK AND TUNNELLING.

Rock Excavation. — Blasting.

The sides of rock excavation are sometimes cut to a steep slope, as one fourth horizontal to one vertical, and sometimes are made perpendicular. Earth, when it occurs over the rock, is of course taken out at the ordinary angle of $1\frac{1}{2}$ to 1, or thereabouts. Rock is excavated to a depth below the grade line sufficient to allow the necessary ballast. The first object in cutting a passage through rock, is to open a working face, so as to get the necessary lines of least resistance.* These lines should, if possible, be at right angles to the beds of stratification, the holes being made parallel with the seams of the rock, when such exist. The powder then tends to lift off the strata. The drill for ordinary work is an octagonal steel bar, from one to two inches in diameter, termed a *jumper*, the chisel end being spread out so as to make a hole somewhat larger than the drill, in order that the latter may work easily. The hole being made, it is cleaned out and dried, water having been introduced to prevent heating of the tool, and filled from one third to half way up with a coarse-grained powder;† after which a wadding of turf, or other light substance, follows, and finally the "tamping" of dried clay, powdered bricks, or stone

* The line of least resistance is that by which the powder finds the least opposition to a vent, at right angles to the length of the drill; or the shortest distance from the charge to the surface of the rock.

† The grains of blasting powder should be of uniform size, from one tenth to one eighth of an inch in diameter, hard, dry, and clean, and burning slower than that for the rifle. A good proportion has been found to be, saltpetre, 65; charcoal, 15; sulphur, 20: the proportion for rifle powder being saltpetre, 75; charcoal, 12.5; sulphur, 12.5.

chips; this material being rammed with a copper bar, which will not strike fire against the rock. Loose sand poured in above the powder has been found to be nearly if not quite as serviceable as the hard driven tamping, the sudden explosion compressing the sand, which by its great friction against the sides of the hole is unable to escape.

Where holes are required in wet places, or in the roof of an excavation, the powder is applied by means of cartridges. The blast in common work is ignited by a safety fuse; but in large operations, especially where it is desirable to fire several charges simultaneously, by electricity; a notable instance of the employment of the latter mode being the great blast at the Chalk cliffs, near Dover, England, where over eight tons of powder were ignited at once, overthrowing 400,000 cubic yards of rock.

Thirty cubic inches of powder weigh one pound. Hence we have the table below, showing the capacity of different drills: —

Diameter of Hole in Inches.	Area in Square Inches.	Ounces of Powder in one Inch deep.	Powder in one Foot deep. lbs.	oz.	Depth of Hole in Inches to contain one lb. of Powder.
1	0.7854	0.419	0	5.028	38.197
$1\frac{1}{2}$	1.7671	0.942	0	11.300	16.976
2	3.1416	1.676	1	4.112	9.549
$2\frac{1}{2}$	4.9087	2.618	1	15.416	6.112
3	7.0686	3.770	2	13.240	4.244

It has been found that one pound of powder will, with a good working face, loosen about four tons of rock; and, generally, that the amount of powder required is as the cube of the line of least resistance. The following charges have been given as the result of actual practice: —

Line of least Resistance in Feet.	Charge of Powder. lbs.	oz.	Line of least Resistance in Feet.	Charge of Powder. lbs.	oz.
1	0	0.50	4	2	0.00
$1\frac{1}{2}$	0	1.75	$4\frac{1}{2}$	2	13.50
2	0	4.00	5	3	14.50
$2\frac{1}{2}$	0	7.75	6	6	12.00
3	0	13.50	7	10	11.50
$3\frac{1}{2}$	1	5.50	8	16	00.00

So much, however, depends upon the character of the rock to be excavated, whether it is hard or soft, stratified or unstratified, and whether the position of the excavation allows of arranging the drills in the most advantageous manner, that the above figures must be regarded as only approximately correct.

The cost of simply *excavating* rock is from four to six times that of earth. An embankment made of rock, the haul being under 500 feet, will cost about $3\frac{1}{2}$ times as much per yard as when made of earth; the haul being from 500 to 1500 feet, from $3\frac{1}{2}$ to 3 times as much; and with a haul of from 1500 to 4000 feet, from 3 to $2\frac{1}{2}$ times as much.

TUNNELS.

Tunnels are driven through hills and spurs of mountains to avoid very deep cutting. At the best they are sources of large expenditure, and should if possible be avoided. When tunnels are cut through rock of a solid and durable character, the roof supports itself; but when in loose or easily decomposed rock, or in earth, an artificial arched lining becomes necessary. Such lining is generally made of brick, especially the arched part, on account of the greater ease of handling and laying brick than stone in so confined a situation. Among the difficulties attendant upon the construction of tunnels are the want of light, air, and drainage. As tunnels generally occur upon summits, or on the approach to them, the latter requirement may be met by the introduction of a light grade. The lower end upon the grade will drain itself; the upper end will require pumps, an occasional well being sunk as low as the contemplated road-bed, or a little lower, to collect the water. Short tunnels may be built by working from the ends only; but as a very limited number of hands can be employed upon so small a working face as the heading affords, when the length becomes considerable, shafts are sunk from the surface to the grade line, and from the bottom of these headings are run in both directions. This operation involves a

large expenditure, as all draining, ventilating, and removal of materials must be effected through the shaft. In cutting a tunnel in rock, a small heading, six or eight feet square, is first taken out by one gang of men, while a larger gang follows in the rear, enlarging the work to the full size, and putting in the masonry when such is required. The speed with which the small opening can be worked is the measure of the progress of the whole ; as the enlarging and lining allows the employment of more hands than the limited dimensions of the heading can accommodate.

After a tunnel has been driven about 500 feet, artificial ventilation becomes necessary. This is accomplished by the ordinary mining expedients, drawing off the bad air by the draught of a chimney, with a fire at the bottom, or forcing fresh air in. If the shaft is made in the bottom of a depression, in order to reduce its length, it may be necessary to provide for the surface drainage in such a locality : regard should be paid in laying out the work to this requirement. The general practice in England has been to multiply the number of shafts ; and in some cases tunnels have been cut entirely through from the shaft headings before the approaches were taken out. In tunnels made through loose material it has frequently been the practice to commence by running forward two small headings, in which the side walls are built, before the remainder of the section is excavated. In other cases the upper part of the tunnel has been opened first, and the arch built ; the space for the side walls being next excavated, and the arch propped by timbers until the walls are built up to connect with it. When the grade descends from a shaft, or at the end of a tunnel where the grade descends into the work, and the water follows the operations in a troublesome manner, the heading may be commenced at the bottom of the intended section, and run along, on a slight ascent, until it reaches the top, when a pit may be sunk to the level of the floor, and the heading may be started again, and the ascent commenced anew.

Examples of Actual Tunnels.

The size and proportions of tunnels may be illustrated best by reference to actual examples. The Box Tunnel, Fig. 32, is between Chippenham and Bath, upon the Great Western Railway, in England. It is 3200 yards in length, 30 feet wide at the

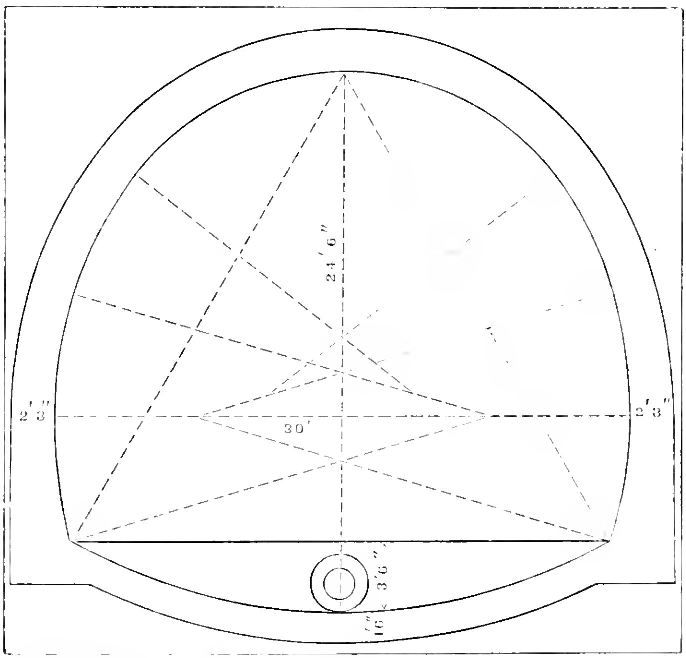


Fig. 32.

widest, and $24\frac{1}{2}$ feet high above the roadway. Nearly half of it passes through the Bath oölite (limestone) and the other half through clay. It is straight, and rises from one end to the other upon a grade of 1 in 100, or 52.8 feet per mile. For about three fourths of a mile it has no lining; the remainder is finished with side walls of the oölite, and an arch of brick. There were seven shafts, having an internal diameter of 25 feet, widened to 30 feet

where they intersect the tunnel. The deepest shaft is about 300 feet. They are lined with brick.

The Brislington Tunnel, No. 3, upon the same railway, 1100 yards long, has nearly the same section as the Box, and is made in a very hard rock (Pennant). This short tunnel had nine shafts in order to hasten the work. A driftway, 7 feet wide and 8 feet high, made in eight months, was run the whole length before the enlarging was commenced. The whole work was done from the inside, on account of the heavy cuts at the ends. The enlarging of the heading was carried on at all of the nine shafts; but the materials were raised up by only three, which were about 110 feet deep, and 15 feet internal diameter.

The Woodhead Tunnel, upon the Manchester, Sheffield, and Lincolnshire Railway, near Manchester, is 3 miles and 26 feet long, 14 feet and 4 inches wide at the level of the rails, and 18 feet and 3 inches high from the rails to the under side of the arch. The sides are vertical, and there is no invert. The road is provided with a small drain, next the wall upon each side. A second tunnel of precisely the same dimensions was afterwards built parallel with it, separated by a longitudinal pier, 21 feet thick, through which are 21 arched openings, about 12 feet wide, connecting the two. The first tunnel was made with shafts, about 10 feet in diameter, upon one side of the centre line, which now descends into the middle of the connecting arches. In making the second tunnel the shafts were not used, but the side walls were first cut through, and openings made horizontally into the line of the new tunnel, all of the material being brought by cross tracks to the finished line, and run out in cars.

The Kilsby Tunnel, upon the London and Birmingham Railway (London and North-western) is about 2400 yards long, with a section of $27 \times 23\frac{1}{2}$ feet. It is cut through clay and sand, and occupied about four years in its construction. It was intended, to make the brick lining 18 inches thick; but this was increased, for the most part, to 27 inches. The whole was laid in Roman cement. During the construction of this work an immense

quicksand was encountered, out of which water was pumped for eight months, night and day, at the rate of 1800 gallons a minute. The large shaft, 60 feet in diameter, and 132 feet deep, was completed in a year: its walls are perpendicular, and 3 feet thick; the bricks being laid in Roman cement. This immense shaft (as well as the second, which was 30 feet less in depth) was built from the top downwards, by excavating for small portions of the wall at a time, from 6 to 12 feet long, and 10 feet deep. The whole number of bricks used in this tunnel was 36,000,000. The cost, which was estimated at £90,000, reached the enormous sum of £350,000.

The Netherton Tunnel, upon a branch of the Birmingham canal, was completed in 1858. It is 3036 yards in length, 27 feet wide, and 24 feet 4 inches high. The brickwork for the lining was generally 1 foot 10½ inches thick in the side walls and arch, and 1 foot 1½ inches in the invert; the thickness being increased where the shafts join the arch, and also in some places where the ground was bad. At several points the invert was pressed up from beneath, in some cases as much as 5 inches, the bricks remaining unbroken. In one place, where the bottom was forced up 8 inches at the centre, and the bricks were crushed, the invert was cut out for a length of 130 feet and rebuilt 1 foot 10 inches thick. This was done in short pieces (about 6 feet) at a time, the side walls being carefully strutted. In rebuilding a portion of this invert, 49 feet in length, the versed sine was increased to 2 feet 6 inches. There were 17 shafts, 9 feet in diameter, 7 of which were left permanently open, and lined with brickwork 9 inches thick. This brickwork was built upon an oak curb 9 × 3 inches, from beneath which the earth was excavated, the curb being temporarily propped until underpinned by the brickwork brought up from the second curb below. The greatest depth of the shafts was 344 feet, and the least 66 feet. The average rate of progress per day of 24 hours, from the commencement to the completion of each shaft, was 2 feet; but counting only the days upon which work was actually done, 3 feet 4 inches. The ma-

terial was chiefly blue marl. The size of the heading was 5 × 3 feet, the bottom being level with the top of the invert.

The Almondsbury Tunnel, near Bristol, upon the Bristol and South Wales Junction Railway, Fig. 33, is 1221 yards long, 18 feet 6 inches wide at the widest part, 17 feet wide at the road-bed, and 19 feet high. It was built for a single track; the whole work being done by means of the shafts, of which there were five; the deepest being 144, and the least 67 feet.

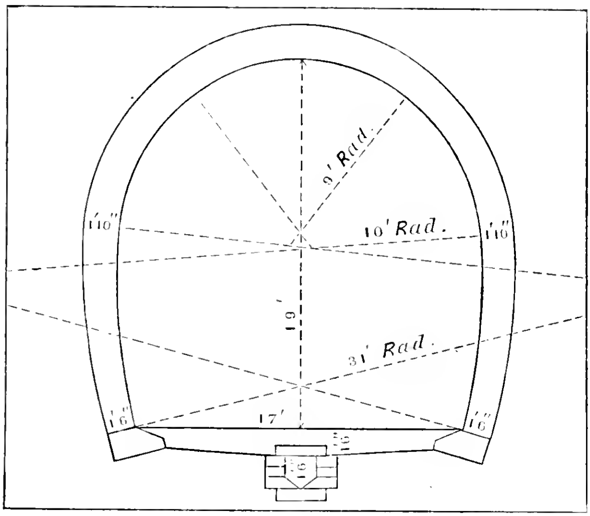


Fig. 33.

The Sydenham Tunnel, on the London, Chatham, and Dover Railway, is 2100 yards long, and is made through the London clay. It had seven shafts, varying from 50 to 186 feet in depth, and 9 feet internal diameter. Two only of these shafts were intended to be left open permanently. The clay in which this work was executed, though yielding freely to the pick, afterwards swelled, and crushed the masonry. The shafts were made 9 feet in diameter, but were afterwards pressed in so as to be hardly more than 6. The headings were 4 × 6 feet, and were run for-

ward at the top, and not the bottom, of the excavation. The original section is shown in Fig. 34. But the swelling of the clay so forced in the masonry that 6780 cubic yards of the side wall and 2065 yards of the invert had to be rebuilt. At the foot of one of the shafts, 120 feet deep, which was the worst place, the tunnel was at first lined with eight rings of bricks, making a

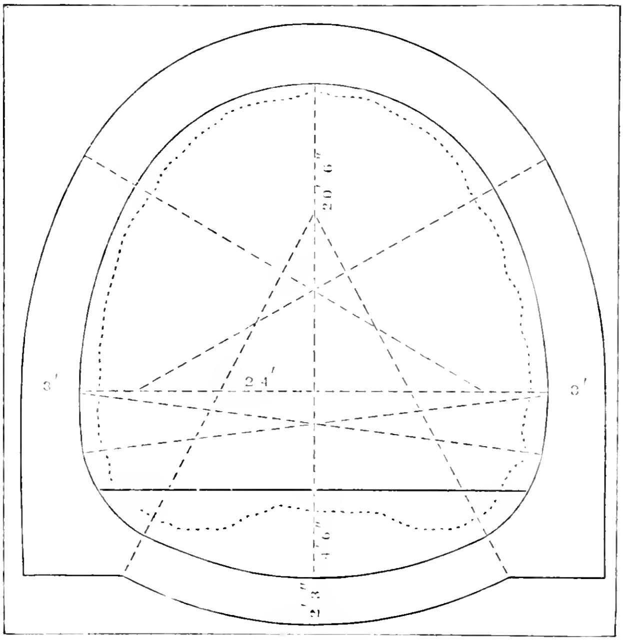


Fig. 34.

thickness of 36 inches. This was cut out and replaced, first by ten, and again by twelve rings of brickwork, or 4 feet 6 inches of thickness; and even some of the last had to be replaced. The shafts had at first a lining of brick 9 inches thick; but this was so pressed in, and the diameter so reduced, as to require a new lining 18 inches in thickness.

This action of the clay begins very soon after the excavation

is made; if it is not noticed within two months, it is no longer feared. The action at the top of the arch is slight, the principal effect being on the invert, which rises first in the centre, and afterwards at the sides. The dotted line in the figure shows the altered shape of the original section. The form was afterwards changed; the first step being to lower the invert, which showed such a tendency to rise, and the next to lower the arch and flatten it at the top, until finally the section of the tunnel was almost a perfect circle, the thickness of the lining above the roadbed being 4 feet 6 inches, and that of the invert 3 feet. Preference was given in this work to lime mortar over cement, as it hardens more slowly and receives the first pressure gradually; by which the bricks do not break so readily.

The Bletchingly and Saltwood Tunnels are between London and Dover, upon the South Eastern Railway The Bletchingly is 24 feet wide, and 21 feet from the top of the rail to the underside of the crown of the arch; the versed sine of the invert being 3 feet, and the form as shown in Fig. 35. The length is 3972 feet; the tunnel being inclined at the rate of 3 feet in a mile. The material through which it is cut is blue clay. The thickness of the lining varies from 1 foot 10½ inches to 3 feet, according to the ground. The time occupied in the construction was 626 days.* The Saltwood Tunnel has the same section as the Bletchingly, except that the invert has 6 inches more versed sine.

* The cost of the Bletchingly Tunnel, reckoning the pound sterling at $5.00, was as follows: —

Materials (Bricks, etc.).	$309.560
Mining the Tunnel and Shafts.	95.045
Labor on the Brickwork.	58.220
Portals, Culverts, Ballast, etc..	34.860
Total.	$497.685

Deducting $21.500 for machinery on hand, when the work was completed, the cost was $476.185. A full description of this tunnel is given in the "*Practical Tunnelling*" of Frederick Walter Simms, the engineer under whose direction the work was executed.

The Hauenstein Tunnel, between Basle and Olten, upon the Central Swiss Railroad, is 2729 yards long, of which 1970 yards were cut from one end. This work passes through limestone, sandstone, and shale. It is made for a double track, being 26 feet wide, 20 feet high above the rails, and of the form shown in Fig. 36. In some places it has an invert, and at others none.

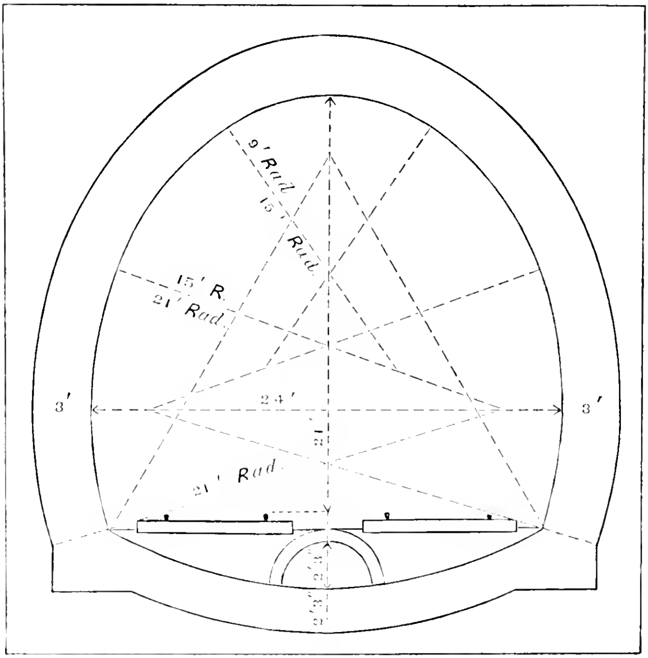


Fig. 35.

The line is straight, and the grade uniform, rising 1 in 38, or 139 feet in a mile. The masonry is limestone, no bricks being used. The heading was about ten feet square, and run forwards at the bottom; but on account of the water which followed the descending grade, it was not kept parallel with the intended roadbed, but was run from the bottom *up*, towards the roof, within the limits of the section. From the heading, enlargements were made at

different places, to get additional working faces. The progress of this work was as follows: Shaft No. 1 was sunk at an average rate of 52 feet per month. Shaft No. 3 at an average of 41 feet per month. The advance in the tunnel from shaft No. 1 was 106 feet per month, at one face. From the south end the rate was $109\frac{1}{3}$ feet per month for 18 months, and 79.7 feet per month

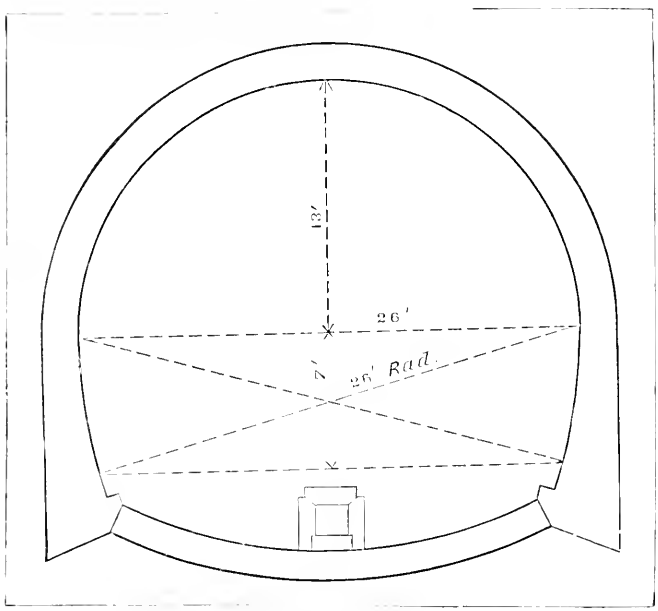


Fig 36.

for 23 months. From the north end the rate was 58.68 feet per month for 19 months, and 56.80 feet per month for the last 10 months. The advance was thus much less at the north end, where the water followed the grade, than at the south end, where it could run off. The whole distance worked from the end faces was 5720 feet in 41 months, or 70 feet a month at one face.

The Mont Cenis Tunnel, which was completed in the summer of 1871, is, without doubt, the largest work of this kind ever undertaken. It is 7 miles and 1044 yards long, and of the section

shown in Fig. 37. The grade rises at the rate of 117.22 feet per mile, or 444.90 feet in all, from the French end to the centre; and falls, for the purpose of drainage, at the rate of 2.64 feet per mile, or 10.04 feet in all, from the centre to the Italian end; thus making the southern portal 435 feet higher than the northern. The tunnel is to be lined with masonry throughout, but with no invert, a covered drain being made in the centre. The side walls are of stone, laid in regular courses, and the arch upon the Italian side of brick. Recesses large enough for several men are

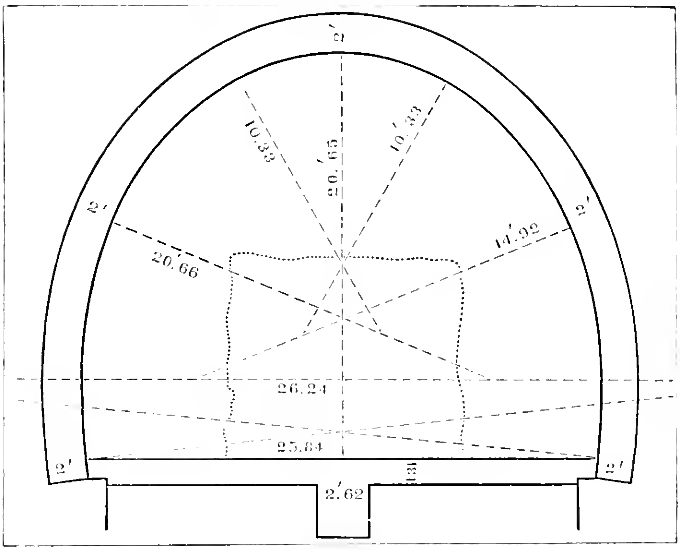


Fig. 37.

made at frequent intervals, and at every 550 yards a tool chamber, ten or twelve feet square, is provided. The side and arch masonry is about two feet thick, though varying at different points. The tunnel has no shafts, being at the deepest more than a mile beneath the summit. The drilling at this great work was done by machinery, the tools being arranged upon a carriage, and driven by a most ingenious application of compressed air, which, after furnishing the required power, escaped, and served

to ventilate the heading. The progress and cost of this tunnel, up to the end of 1868, were as follows:—

Year.	Advance in Feet per Year.	Whole advance in Feet	Expense in Dollars.
1857 } 1858 }	1,632	1,632	673,800
1859	1,211	2,843	326,000
1860	1,126	3,969	600,000
1861	1,191	5,160	500,000
1862	2,043	7,203	400,000
1863	2,631	9,834	700,000
1864	3,568	13,402	1,310,400
1865	4,014	17,416	1,100,400
1866	3,362	20,778	1,128,800
1867	4,960	25,738	1,200,000
1868	4,330	30,068	1,500,000

This gives a total advance of 30,068 feet in twelve years, or 2506 feet in a year, or 209 feet a month, or 104.5 feet a month at each working face. The whole expenditure above is $9,439,400; or $786,616 a year.

The Nerthe Tunnel, upon the railway from Marseilles to Avignon, is 5072 yards long, 26 feet 3 inches wide at the widest part, and 24 feet 7 inches high above the rails. In its construction twenty-four circular shafts, 10 feet in diameter, placed 30 feet on one side of the tunnel, and communicating with it by lateral galleries 10 feet wide, were employed. The work runs through marl, lias, dolomite, and other limestones. Of the whole length, 950 yards are unarched; the remainder has a lining 2 feet 9 inches thick. It was completed in three years, at a cost of about $400 per running yard. The following figures show the cost of sinking the several shafts, not including the cost of the wooden curbs or the masonry.

Number of the Shaft.	Depth in Feet.	Average Cost per Foot.	Cost per Foot for a Depth in Feet of 0 to 197.	197 to 328.	328 to 459.	459 to 610.
1 to 5	67	$8.88	$8.88	....	....	....
6	200	10.97	10.97	....	....	....
7	178	12.80	12.80	....	....	....
8	252	13.72	13.47	$14.70	....	....
9	345	16.46	15.06	18.41	....	....
10	292	15.85	15.24	17.31	....	....
11	387	16.77	15.24	18.29	$19.81	....
12	482	16.77	15.55	18.60	21.34	....
13	552	16.77	13.05	16.10	19.76	$22.70
14	623	16.77	11.58	14.63	18.29	24.06
15	519	18.29	13.60	16.64	25.91	....
16	472	15.85	13.05	16.10	19.81	....
17	469	15.85	13.29	16.34	19.69	....
18	375	15.24	13.78	16.83	18.29	....
19	395	12.19	10.42	13.47	....	....
20	489	13.41	10.97	14.02	15.24	....
21	467	18.29	15.73	18.78	16.02	....
22	361	15.24	13.78	16.83	....	....
23	299	15.85	14.94	17.51	....	....
24	258	13.41	13.41	13.41	....	....

In the Journal of the Franklin Institute for 1841 may be found a description of the Chesapeake and Ohio Canal Tunnel at the Pawpaw bend of the Potomac River, taken from the report of Ellwood Morris, Esq., the engineer. This work is 3118 feet long, through a slate rock which, though firm enough to hold up the roof, scaled off badly. It was thus deemed advisable to put in a light, well-packed arch, the side walls being 13 inches, or a brick and a half thick, and the arch, which was a semicircle of 24 feet span, 9 inches, or one brick thick. This lining, requiring in all 3,500,000 bricks, Mr. Morris proposed to complete in a year, by working as follows: First, to carry up the side walls to the springing. Second, in sections of 500 feet, by reverse moulds and without centring, to carry up the arch on both sides to

the angle of repose, or by bringing into play the coherence of cement, up to 45° or 47°.* Third, by a system of detached centres, framed to leave open about 30° of the crown, each supporting 3 feet lineal of the arch, and leaving an interval of 4 or more feet to be supported by the cohesion of the cement, to carry up the spandrels to about 75°, or within 15° of the crown on each side. Fourth, by a very light centre (handled by two men) to keep up the crown, in sections of 2 feet, shifting the crown centres as each successive section is keyed up and packed. Thus, by working in long reaches, course by course, the cement would set in one part while the workmen were engaged at another. The keying up, it was reckoned, could be done at the rate of 10 lineal feet per day, working from one point only.

The Kingwood Tunnel, upon the Baltimore and Ohio Railroad, 4120 feet long, 22 feet wide, and about 20 feet high, was driven through a series of slates and other stratified rocks, in some places very compact, at an average advance of 104 feet per month for 1040 feet at the western end, at 45 feet per month for 315 feet at the eastern end, at 55 feet per month for 820 feet at shaft No. 1, at 49 feet per month for 735 feet at shaft No. 2, and at 64 feet per month for 1210 feet at shaft No. 3; the average from the ends being 79.7 feet per month, and from the shafts 56.43 feet. The progress in sinking the shafts, which were about 170 feet deep, and descended through sandstone and slates in the upper and middle portions, and hard slate rock at the bottom, averaged 25 feet per month, though the last month's work at the bottom amounted to 47 feet. The side walls of the Kingwood Tunnel are of stone, and the arch, which is very nearly a semicircle, is in some parts of stone, in some of brick, and in others of iron and masonry combined. The natural rock at some points

* Mr. Morris refers to the experiments of Pasley and of Brunel, who found that from 20 to 30 bricks could be held horizontally by cement.

caved in badly overhead. To check this, in a manner at once speedy and safe for the workmen, sections of an arch, 3 feet in length, made of cast iron an inch thick and stiffened with two vertical ribs 6 inches deep and an inch thick, placed 2 feet 4 inches apart upon the under side of the arch, were made in two segments of 90° each, and carried into the tunnel upon a frame so arranged on a car that the segments could be raised up and spread out until their feet rested upon the side walls, where they were securely bolted together by flanges at the crown, forming thus a semicircular arch of the required size, from which the roof was propped up, and upon which a rough arch of stone was turned, the vacant space above, in some cases 25 or 30 feet, being closely packed.

Rate of Progress in Tunnelling.

The following additional facts furnish the means of judging to some extent of the rate of progress that may be expected upon this class of works. A tunnel upon the Cincinnati and Dayton Railroad, about 10,000 feet in length, through marl, clay, and limestone, advanced during the year 1853, from the north end, 53.5 feet per month, for 368 feet; from the south end, 65 feet per month, for 268 feet; the average being $59\frac{1}{4}$ feet. From shaft No 1, the advance was 67 feet per month, for 336 feet; from shaft No. 2, 67 feet per month, for 336 feet; and from shaft No. 3, 60 feet per month, for 300 feet. The shafts for this work, three in number, 152, 194, and 199 feet deep severally, and elliptical in section, 12 × 20 feet, were sunk 35 feet per month.*

The Black Rock Tunnel, 1932 feet long, was driven through hard metamorphic slates at an average rate, from the ends, of 40.5 feet per month, and from the shafts of 33 feet. At the

* The above progress was made with a light force. The engineer of the work states that a full gang advanced each of the headings, for a short time, 89 feet per month; and that, with sufficient force, the shafts might have been sunk at the rate of 80 feet per month.

Pulpit Rock Tunnel, 1638 feet long, in hard sandstone, with veins of quartz, the heading at the eastern end advanced 47 feet, and the shaft headings 34 feet, per month. A tunnel in Georgia, upon the Western and Atlantic Railroad, 1477 feet long, through clay and sandstone, advanced 39 feet per month, for 600 feet, at the western end, and 59 feet per month, for 847 feet, at the eastern end. The Blue Ridge Tunnel, in Virginia, 4273 feet long, through blue slate, trap, and quartz, advanced at the rate of $27\frac{3}{4}$ feet per month at the western end, and $29\frac{1}{3}$ feet at the eastern end.

Rate of Progress in sinking Shafts.

With regard to the time consumed in sinking shafts, the Pemberton pit in Sutherland, Scotland, 1800 feet deep and 12 feet in diameter, was sunk in seven years. A shaft at the Wearmouth colliery, 1578 feet deep, was sunk in ten years. A well 207 feet deep was sunk at Fort Regent, Jersey, through compact sienite, at an average of 9.4 feet per month. A shaft at Tamaqua, Penn., was sunk 350 feet, through a hard rock, at 20 feet per month. Another, at Pottsville, 160 feet deep, through hard rock, was sunk 75 feet per month. A shaft of the Bletchingly Tunnel, $10\frac{1}{2}$ feet diameter, was sunk at the rate of 90 feet per month, for 51 feet, through clay and shale. The shafts of the St. Martin's Tunnel, upon the Bourbonnais Railway, from 70 to 176 feet deep, were sunk through a hard porphyritic rock, at from 12 to 20 feet per month.

We may, from the preceding facts, conclude, that, under ordinary conditions, in an average quality of rock, and with hand labor, a monthly advance of 50 feet at each end of the tunnel may be counted upon, and an advance of 30 feet per month each way from the bottom of the shafts. By machinery these rates may be doubled. Shafts of average depth, and through any rocks except the very compact unstratified ones, may be sunk at the rate of 25 feet per month. The full size of the tunnel, or the shaft, has little to do with the progress, as the larger sections furnish a

larger working face. Except at the Mont Cenis and Hoosac Tunnels, machinery has not been employed to any extent in drilling. The saving made by power drills over hand work is a saving of time, but not of money, except so far as the interest upon the outlay is concerned. When the heading has been made, enlargements at numerous points will furnish room for the employment of increased forces.

Drainage and Ventilation of Tunnels.

The drainage of a tunnel is effected either by a covered passage under the roadbed at the centre, or by open ditches at the sides. Small drains from the back of the side walls, leading through to the roadbed drains, have, in some cases, been found of service. It has been a practice in some parts of Europe to coat the masonry with cement or bitumen, to prevent the water from leaking through, and keeping the roadway wet. The sleepers are not found to decay, as a general thing, faster in tunnels than in the open air: indeed, it has been stated that they last better than when exposed to the sun and rain. Artificial ventilation is not found necessary in completed tunnels of less than three miles in length; and in many cases the shafts have been considered rather to retard than to aid the clearing of the smoke. An engine run quickly through appears to be the best ventilator; the air is driven in advance of the train, and rushes in behind it.

General Remarks.

The general practice in Europe is to allow only one train to be in a tunnel at one time. Many of the short works of this kind in France are built upon curves. The Hoffmuhl, 810 feet long, and the Bourg la Reine, 669 feet, are upon curves of about 2500 feet radius. The Fesc, 547 feet, has a curve of 800 feet radius; and the Vierzon, 682 feet long, is upon a reversed curve of 3280 and 4100 feet radii. These works, however, are all quite short.

It may occur that a shaft can be driven into a heading horizontally, from a hill-side, at less expense than vertically, from above. Some of the small side-hill tunnels upon the French approach to Mont Cenis had such shafts. The cars upon American roads are now made so wide, and require so much space between the tracks, that the width of roadbed, originally ample, is at the present time only sufficient to allow the trains to pass safely. Fig. 38, which is a section of the Spruce Creek Tunnel, on the Penn-

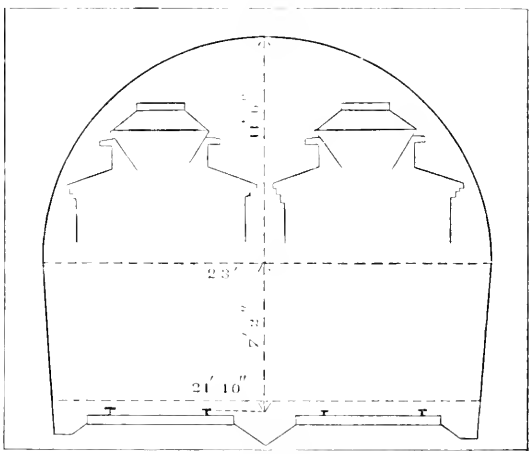


Fig. 38.

sylvania Railroad, shows how little room there is to spare when two trains occupy the tracks: the width in this case is 23 feet at the widest part, the gauge 4 feet 9 inches, and the width between tracks 6 feet. The Alleghany, or Gallitzin Tunnel, upon the same road, has a width of 24 feet.

The Hoosac Tunnel.

The Hoosac Tunnel, now in process of construction beneath the mountain of the same name in Western Massachusetts, is 25,031 feet, or $4\frac{3}{4}$ miles in length. It is to be of the form and

dimensions shown in Figs. 39, 40, and 41, Fig. 39 representing the section in the solid rock where a lining is not required. The right hand half of Fig. 40 shows a half section of the tunnel arched; the left hand half of the same figure shows a half section with the lining and with a preliminary timber support to the roof to protect the workmen until the brick arch is completed. Fig. 41 shows the section of the lining where an invert is required. The grade rises at the rate of $26\frac{40}{100}$ feet per mile from

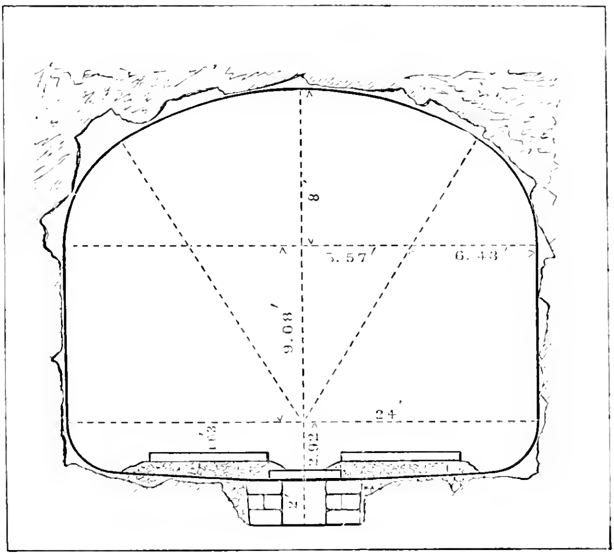


Fig. 39.

each end to the centre. Shaft No. 1 is 1498 feet from the western end, and has a section of 6 × 6, and a depth of 215 feet. Shaft No. 2 is 2183 feet from the western end, and has a section of 13 × 6, and a depth of 277 feet. These shafts were sunk for the purpose of drainage, before the heading had been driven through from the west end. The "West Shaft," 2447 feet from the western end, is 14 × 8 feet, and 318 feet deep, and is the working shaft where the material excavated eastward is hoisted.

The "Central Shaft," about midway between the ends of the tunnel, is elliptical in section, 27 feet in diameter along the line of the road, 15 feet in diameter across the road, and 1030 feet deep.

The Hoosac Mountain, on the line of the tunnel, rises to a height of 2500 feet above the sea, and about 1800 feet above the road. There is a double summit, and the central shaft is located

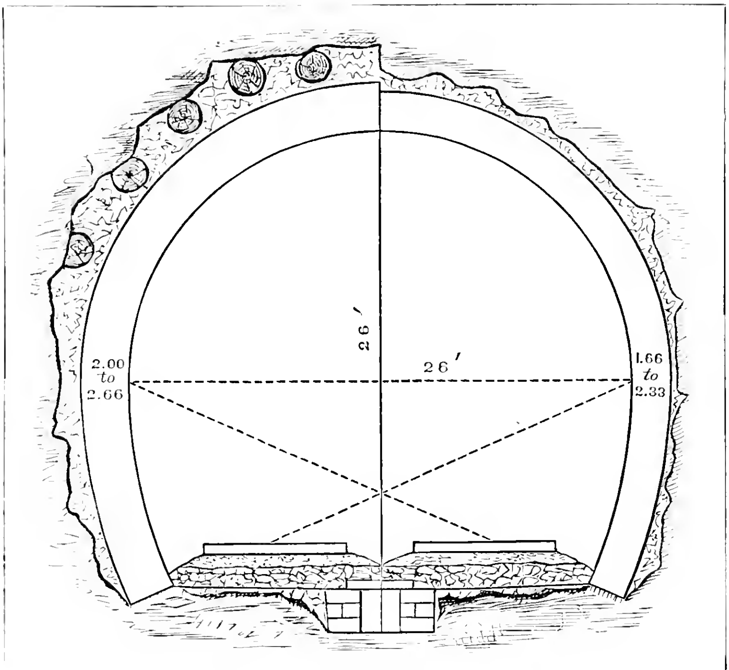


Fig. 40.

in the intermediate depression. The general character of the rock at the eastern end, and at the central shaft, is mica slate, with quartz; and at the "West Shaft," a hard quartzite. The first 2000 feet at the western end, being through a rapidly decomposing material, required to be arched with brick. The headings have been driven, and much of the enlarging done, by the machine known as the Burleigh Rock Drill. These drills are in

each heading mounted upon two carriages, standing side by side, with a space of about 6 feet between them. Each carriage supports five drills; but seldom more than four (eight in all) are in motion at one time. They make from 180 to 260 strokes a minute, with a pressure of 60 pounds per inch. When a blast is to

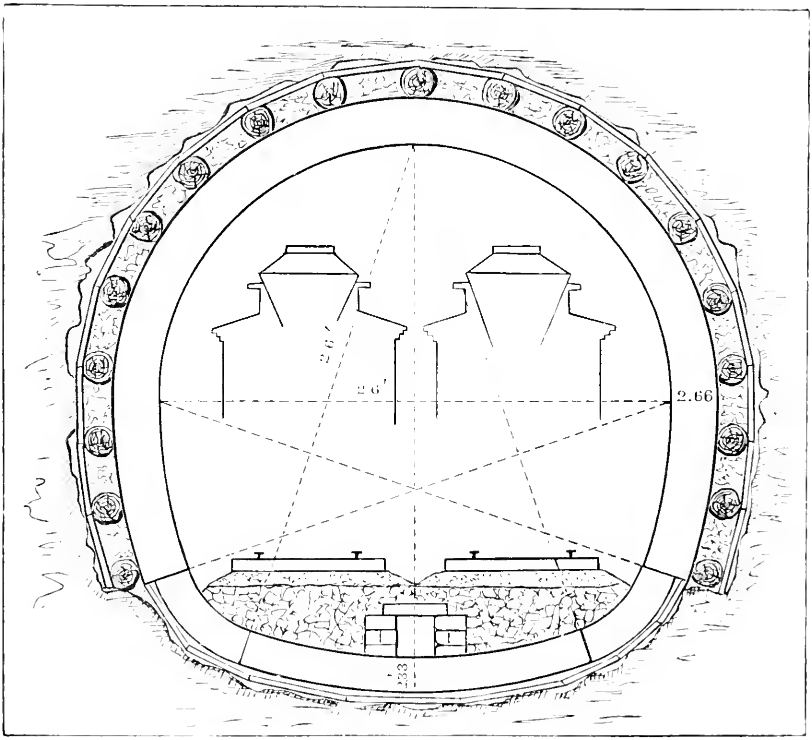


Fig. 41.

be fired these carriages are run back out of the way. The power for working the drills is the same as that used at the Mont Cenis Tunnel, viz., compressed air. At the eastern end of the work a dam across the Deerfield River gives a head of 21 feet, which works four turbine wheels, each wheel working four air cylinders. The air, compressed to a tension of about 65 pounds per inch, is

conveyed into the tunnel through two eight-inch cast iron pipes; and experience shows that it loses little or nothing of its force in the transit. When the gauge at the compressors shows 65 pounds, that at the heading shows 63 pounds.

At the "West Shaft" the air for working the drills and ventilating the tunnel is compressed wholly by steam. There are five machines, each being steam engine and compressor combined, having two cylinders 12 × 12 for compressing, and one steam cylinder 9 inches diameter by 18 inches stroke. Each of these machines will drive two drills, each drill making a two-inch hole at the rate of 240 strokes a minute. The hoisting engine is 40 horse power, and connected with the elevators in the shaft by steel wire ropes $1\frac{1}{4}$ inches in diameter. The material excavated at the bottom of the shaft is raised to the surface by means of a double elevator or "lift." Small cars, carrying about half a yard each, are run between the headings and the shaft, there being a double line of rails for up and down trains. The cars are run directly upon the floor of the lift, which takes them with their load bodily to the surface, when they are run off upon the track to the dump and tipped. The empty car then returns to the elevator, which at once descends, while at the same time, and by the same motion of the engine, its *twin*, standing back to back to it, comes up on the opposite side of the shaft, bringing its loaded car. In this way a car load of rock comes to the surface every four minutes. The same method will be adopted at the central shaft when it reaches the grade line. At the eastern end the material is drawn out by a small locomotive, which can be run directly into the heading.

At the eastern end the excavation of the heading is effected by ordinary cannon powder, and the enlarging by nitro-glycerine. At the western end, where the rock is very much harder, nitro-glycerine is almost wholly used; and the central shaft has been sunk altogether with the same material. About 6000 pounds of this new explosive, and 250 kegs (25 pounds each) of powder, are used every month. The explosion is effected wholly by electricity,

every charge in a blast, sometimes as many as 30, being discharged simultaneously. The underground work is carried on in three shifts daily, eight hours being a day's work; thus the tunnelling goes on unceasingly through the twenty-four hours, Sunday only excepted.*

The preceding sections are all drawn to the same scale, and thus show the correct relative size of the various tunnels referred to.

The centres used at the Bletchingly Tunnel will be illustrated in a following chapter.

* The condition and rate of progress of this work upon May 1, 1870, was as follows: The heading at the eastern end was driven 6976 feet, and the tunnel was completed for 2500 feet. At the western end the heading was driven 4860 feet, and the tunnel completed 2524 feet. The central shaft was sunk to a depth of 920 feet. The rate of progress at the same time at the eastern end was 132 feet of heading per month, and 250 feet of completing. At the western end the heading was advancing 100 feet per month, and the enlarging and completing 170 feet. Upon January 1, 1871, the east heading had reached 8036 feet, the west heading 5708 feet, the central shaft had reached the grade, and headings to the east and west of 60 and 87 feet respectively had been driven, making an aggregate of headings done upon January 1, 1871, of 13,890 feet, and leaving 11,141 feet to be penetrated. The distance penetrated at the east heading in 1869 was 1239 feet; in 1870, 1514 feet. The shaft was sunk 215 feet in 1869, and 230 feet in 1870, going down for a part of the time at the rate of 30 feet a month. During the month of April, 1872, the east end was advanced 131 feet, and the west end 140 feet. The heading from the central shaft in the same time advanced 98 feet in each direction. The total length opened upon May 1, 1872, was 18,971 feet; leaving 6060 feet to be completed. The time fixed by contract for the completion of the work was March 1, 1874. The headings from the eastern end and from the central shaft met upon the 12th of December, 1872, and the headings from the western end and from the central shaft upon the 27th of November, 1873. The first train passed through the tunnel upon the 9th of February, 1875.

The reader will perceive that the preceding remarks, relating to the Hoosac Tunnel, were written during the progress of the work. It is not considered necessary, however, to change the text now that the work is done.

CHAPTER VII.

STRENGTH OF MATERIALS.—EXPERIMENTAL DATA.

General Remarks upon the Resistance of Materials.

The materials employed in the construction of bridges are steel and wrought iron, in the various forms of rods, bars, and plates, cast iron and wood. These materials are subjected to a variety of forces, according to the positions they occupy in the structures of which they form a part. There are five distinct strains to which a piece of wood or metal may be exposed, each of which tends to destroy the material in a different manner. A beam or bar may be pulled apart by stretching — *Tension:* it may be destroyed by crushing — *Compression:* it may be broken across the length — *Transverse Strain:* it may be cut or crushed across — *Shearing* or *Detrusion:* or it may be twisted asunder by *Torsion.* These several strains may act singly or in combination.

For all practical purposes the materials used in bridge building may be regarded as susceptible of extension and compression, to a degree which is directly as the force producing the distortion. The elastic reaction of the material is also proportional to the increase or decrease of length which is impressed upon it. Moreover, so long as any substance may be considered as perfectly elastic, the force producing a given compression is the same as that producing an equal extension. A bar of wrought iron an inch square is extended $\frac{1}{10000}$ of its length by a ton of direct tensile strain, $\frac{2}{10000}$ by two tons, $\frac{3}{10000}$ by three tons, and so on, up to a strain of about twelve tons per square inch: and when extended $\frac{3}{10000}$ of its length, it will be reacting with three times the energy that it will when extended $\frac{1}{10000}$. This

is the condition of perfect elasticity. When a body is strained beyond a certain limit it receives a permanent distortion, or, as it is termed, a *set*. This *set*, however, is not increased by a subsequent application of the same force that produced it. If we place five tons upon a cubic inch of wrought iron it is compressed $\frac{5}{10000}$ of an inch, and it takes a certain permanent set due to this compression. On the removal of the five tons it partly recovers itself, but it never returns to its original height. The cube is in a new condition. The subsequent replacing and removal of the five tons any number of times, after this first set, never alters these new conditions nor increases the permanent distortion. Time, however, is an element in this operation. A locomotive run on to a bridge will produce a certain depression; first, by the closing up and stretching of the joints, and second, by the elongation and compression of the material. If the load is left on for 24 hours the depression will be seen to have increased, but after a longer time, perhaps several days, the settling will cease, the fibres having adjusted themselves to the new conditions impressed upon them. When the load is removed the bridge will partially regain its first position, owing to the elasticity of the fibres, and a subsequent repetition of the same load will not depress it more than the first application.* Some doubt has been thrown upon the correctness of the above statement, on account of certain experiments in which a second application of the force appeared to increase the permanent depression. It is probable, however, that in such cases the full set was not really attained at first, on account of the time during which the load

* In his fine work upon the Britannia and Conway Tubular Bridges, Mr. Clark observes: "Time is an important element in producing the ultimate permanent set in any elastic material: but when the permanent set due to the strain is once attained, the continuance of the same strain induces no farther deflection: which is confirmed by the fact that no subsequent change has occurred in the deflection of the Conway Bridge from two years of use, nor has any increase in the versed sine of the Menai Suspension Bridge taken place in twenty-five years, where the strain is greater than in the plates of the Conway Bridge."

was applied being too short. Ample time, too, should be allowed for the material to recover itself after the removal of the load before determining the set.* The fact that the permanent set is not increased by a subsequent application of the force that produced it, has led in some cases to compressing or extending the bars about to be used in building a bridge before they are introduced into the structure.

Experiment has determined what has been called the coefficient, or modulus of elasticity; this being the number of pounds per square inch which would extend or compress a bar by an amount equal to its own length, supposing that the law of elasticity held good for so great a range. Thus, since one ton extends an inch bar of wrought iron $\frac{1}{10000}$ of its length, at the same rate 10,000 tons would extend it through its whole length; and 10,000 tons, or 22,400,000 pounds, is thus the modulus of elasticity for wrought iron. Another term sometimes used is the modulus of elasticity *in feet;* by which is denoted the length of a bar required to produce the same strain as above given in pounds. In the case of wrought iron, as an inch bar weighs 3.3 pounds per foot, the modulus of elasticity in feet is 22,400,000, divided by 3.3, or 6,787,878 feet.

The resistance to tension, to compression (as regards simple crushing), and to shearing, is in proportion to the sectional area; if we double the area we double the strength. The resistance to a cross strain is *as the breadth, inversely as the length, and as the square of the depth:* if we double the breadth, we double the strength; if we double the length, we divide the strength by two; and if we double the depth, we multiply the strength by four.

Whatever depression takes place in a beam placed horizontally and supported at the ends, tends to shorten the upper and to

* Mr. Stoney remarks (Theory of Strains, Vol. II., p. 303) that "the residual set, after the strain has been removed, also takes *time* to adjust itself to a permanent condition; and some crude experiments of my own tend to prove that the set of wrought iron relaxes to a considerable extent, even after the lapse of several days after the strain has been removed."

extend the under side, the fibres of the top part suffering compression, and those of the under-side extension, while between the top and bottom lies a plane (termed the neutral plane), which is neither compressed nor extended, but which serves to connect the top and bottom, and thus plays an important part in resisting the flexure. Thus, in beams subjected to a cross strain, the resistance is effected by the opposition of the fibres to compression and extension.

Upon the Absolute Strength of Timber.

With regard to the absolute resistance of the various materials employed in bridge building a large amount of experimental evidence has been obtained, an abstract of which, sufficient for our present use, we proceed to give.*

Although a great variety of woods have been experimented upon, the results of practical value may be stated very briefly, inasmuch as not more than two or three out of the long list generally given in works upon the strength of materials are in common use. The following table is from the best among the numerous authorities: —

* The reader who is desirous of consulting the original authorities upon this important subject is referred to the following works: Barlow on the Strength of Materials; Tredgold's Carpentry; Fairbairn on the Application of Cast and Wrought Iron to Building Purposes, and Useful Information for Engineers, 1st and 2d series; Hodgkinson's Experimental Researches on the Strength and other Properties of Cast Iron; Report of the Commissioners Appointed to Inquire into the Application of Iron to Railway Structures; Mr. Edwin Clark's work upon the Britannia and Conway Tubular Bridges; Reports of Experiments on the Strength and other Properties of Metals for Cannons, by Officers of the Ordnance Department, U. S. Army; Murray on Ship-building in Iron and Wood; *Résistance des Matériaux*, by General Morin; *Résumé des Leçons sur l'Application de la Mécanique à l'établissement des Constructions*, by Navier; Experiments upon Wrought Iron and Steel, by David Kirkalday. The article on The Strength of Materials in the 8th edition of the Encycl. Brit., Chapters IV. and V., of Professor Rankine's Civil Engineering, and the first half of the 2d volume of Mr. Stoney's work upon the Theory of Strains in Girders, also give valuable abstracts.

Tensile Strength, in Pounds per Square Inch, according to Mr. Barlow.		Compressive Strength, in Pounds per Inch, according to Mr. Hodgkinson	
Riga Fir,	12,600	Fir,	6,499
Mar Forest Fir,	12,000	Red Deal,	5,748
New England Fir,	12,000	White Deal,	6,781
Red Pine,	10,000	Red Pine,	5,395
Pitch Pine,	10,500	Pitch Pine,	6,790
Norway Pine,	12,000	Yellow Pine,	5,375
English Oak,	10,000	English Oak,	6,484
Dantzic Oak,	14,500	Dantzic Oak,	7,731
Canadian Oak,	12,000	Quebec Oak,	4,231

The figures given by different experimenters vary widely on account of the more or less perfect condition of the wood tested. Timber, when well seasoned and dry, will bear double the compression that it will when green or wet. The difference between the strength of oak and pine, as regards tension and compression, appears from the above figures to be a little in favor of the hard wood.

The lateral adhesion of the fibres, or the resistance offered to being pulled apart sidewise, is, according to Tredgold, for pine, spruce, and fir, from 500 to 1000 pounds, and for oak, 2300 pounds per square inch.

The resistance of seasoned oak treenails to shearing, or cutting across the fibre, is shown by the following table, prepared from the data given in Mr. Murray's work on ship-building, referred to in the note upon a preceding page :—

Diam. of the Pin in In.	Area of the Pin Sq. In	Shearing Strength in Pounds in a 3 Inch Plank.	Shearing Strength in Pounds in a 6 Inch Plank.	Shearing Strength per Sq. Inch in a Plank 3 In. thick.	Shearing Strength per Sq. Inch in a Plank 6 In. thick.
1	0.78	3340	3540	4282	4538
$1\frac{1}{4}$	1.23	5080	5080	4130	4130
$1\frac{1}{2}$	1.77	6080	6920	3435	3910
$1\frac{3}{4}$	2.41	7720	9560	3203	3967

From this it would seem that small treenails have a higher unit of resistance than large ones. Great care should be taken in seasoning treenails, especially when it is done artificially, not to make them so dry as in any way to damage the fibre.

The shearing strength of fir, in the direction of the grain, is, according to Barlow, 592 pounds per square inch. Mr. Rankine gives the following figures (page 769, Civ. Eng.) for the same strain, which, as he remarks, is very nearly the same as the lateral adhesion, or the tenacity across the grain :—

Fir (Red Pine), pounds per inch, . .	500 to 800
Spruce, ...	600
Larch, ...	970 to 1700
Oak, ...	2300
Elm and Ash, ...	1400

With regard to transverse, or cross strength, the hard woods would seem to be somewhat stronger than the soft ones. This may arise from the superior resistance to extension and compression in the fibres of the hard woods, or from the greater lateral adhesion, or from both combined. Mr. Stoney, in Vol. I. of his work upon the Theory of Strains, gives the following coefficients of rupture for a transverse strain, as determined by various experimenters, the numbers being the breaking weight in pounds for a semi-girder, of which the length, breadth, and depth are each one inch, the girder being fixed at one end, and loaded at the other.

Spruce, ...	1346
American White Pine, ...	1229
American Red Pine, ...	1527
Pitch Pine, ...	1727
English Oak, ...	1694
American Red Oak, ...	1687
American White Oak, ...	1743
American Live Oak, ...	1862

It is to be borne in mind, when considering the superior strength of oak, that it is 50 per cent. heavier than pine, and thus that a larger permanent strain is kept upon the fibres of oak by its own weight, which, to some extent, neutralizes its extra stiffness.*

Upon the Nature and Preservation of Timber.

The best timber is that which has grown slowly, upon a soil rather dry than moist, and is compact and heavy, the annual rings being thin and uniform, showing a hard, clear surface when cut, and not a dull or chalky one, free from clefts or radial cracks, and from cup shakes or cracks between the annual layers. Timber is best when cut at or near the maturity of the tree, as a young tree has too much sap wood, and an old one is likely to get hard and brittle at the core. Probably 50 years is the least, and 100 years the greatest age at which the ordinary kinds of wood should be cut. The best season for felling is when the sap is quiet, or in midsummer and midwinter. Timber exposed freely to the air in a dry place, sheltered from the sun and rain, requires two years and upwards, according to the size, to become well seasoned. A small part, however, of the material put into our public works receives sufficiently careful treatment in this respect, and hence the short life of the majority of wooden structures. Artificial seasoning is effected by exposing the material, properly piled in a suitable building, to a current of hot dry air. Timber loses in drying from 15 to 30 per cent. of its weight, and shrinks, across the grain, from 2 to 5 per cent. Wood lasts the best when kept dry and well ventilated. When kept constantly wet it is somewhat

* Mr. Haupt found, by experiments made upon pieces of wood 5 feet long, 3 inches deep, and one inch wide, supported at one foot from the end, the load acting with a leverage of four feet, that while white and yellow pine were injured so as not to recover their shape by a strain of from 2200 to 2800 pounds per square inch, applied from 5 to 10 minutes, white oak was not injured by a strain of 4248 pounds for 15 minutes, or by 3648 pounds per inch for 40 hours. — *Bridge Construction*, p. 61.

softened, and will not resist so much, but it does not decay. Piles placed in the Rhine nearly 2000 years ago have been found quite sound during the present century, and the roof timbers of some of the older Italian churches are still in good condition. Many highway bridges in this country are apparently uninjured by from 40 to 50 years of use; and railway bridges, made of good material, and carefully protected, have been in service for 20 years, and are yet in good order.

Wood decays the fastest when alternately wet and dry, or when subjected to a hot, moist, close atmosphere. Thorough seasoning, protection from the sun and rain, and the free circulation of air, are the essentials to the preservation of timber. Oil paint will protect wood from moisture from without, but unless it is perfectly dry when painted, the moisture within will be unable to escape, and will cause decay. Several different methods of preservation are now in use, consisting of an injection of different chemical preparations into the pores. Chapman's process employs sulphate of iron (copperas); Kyan's process, corrosive sublimate (bichloride of mercury); Burnett's process, chloride of zinc; Boucherie's method, sulphate of copper; and Mr. Bethell saturates the timber with creosote. In these several operations the air is exhausted from the tank in which the timber is placed, the sap drawn out from the pores, and the solution forced in.*

The woody fibre is seen by the microscope to consist of long, slender tubes, upon the tenacity of which depends the tensile strength of the timber. The lateral adhesion, or the strength across the grain, depends upon the adhesion of the sides of the tubes. The pines or cone-bearing trees have a straight and regular fibre, and are well adapted to direct tensile strains; but the lateral adhesion is small, so that they are much more easily split along the grain, and much less suitable to resist shearing endwise, or sliding of the fibres on each other, than the hard woods.

* A full description of the several solutions employed in the preservation of timber, and a description of the apparatus, and of the mode of proceeding, will be found in Messrs. Colburn and Holley's "Permanent Way of European Railways."

Upon the Nature and Strength of Iron.

Iron is employed by the engineer in three several forms: cast iron, wrought iron, and steel. Cast iron is known as charcoal, coke, anthracite, or bituminous, hot or cold blast, foundery or forge iron, according as it is made with one or other of the several fuels, with the hot or cold blast, and whether intended for foundery use or for making into wrought iron. Of cast iron there are two principal kinds, the gray and the white, differing in their chemical and physical characters, and between these two are several intermediate varieties, which resemble more or less the gray or the white, as they approach nearer to one or the other. The gray iron contains one per cent., or even less, of carbon, chemically combined, and from one to four per cent. of carbon in the state of plumbago, mechanically mixed. The white iron contains from two to five per cent. of carbon, in a state of chemical combination. The gray iron is soft and tough, slightly malleable when cold, may be drilled, planed, or turned, melts at a lower heat than the white, being red when molten, remains fluid a long time, fills the mould readily, and gives fine sharp angles to the casting. The fracture is granular, of a gray color, with a metallic lustre. White cast iron is hard, brittle, and sonorous, cannot be worked, is not easily melted, is white when fluid, thickens rapidly, and shows a white crystalline fracture, with a vitreous lustre. The gray iron is most suitable for strength, the white for hardness. The two varieties may be produced from the same ore under different conditions of temperature. The carbon requires to cool slowly in order to form graphite or plumbago, and to exist as a separate material in the iron; rapidly cooled, the carbon remains chemically combined, thus producing white iron. The process of "chilling" where the outer surface of a casting for a half or three fourths of an inch is an exceedingly hard white iron, while the interior of the mass will be tough and gray, illustrates the effect of sudden cooling. The hard "skin" of all castings, due to the rapid cooling of the surface, is another illustration of the same

process. A bright-gray fracture, uniform in color and in structure, with a medium-sized grain, sharp and rough to the touch, shows a good tough iron. As the color becomes darker and the lustre less, the iron becomes soft and weak. As the fracture becomes lighter the iron is harder, but less tough, until the color is white, and the lustre vitreous, when it is too hard and brittle for use, except in combination with softer irons. If the edge of a casting is indented by a blow from a hammer, the iron is tough and slightly malleable: the edge of a brittle casting will break. Besides the general division into gray and white iron, the several varieties are numbered by the trade according to the hardness, or the amount and quality of the carbon contained. No. 1 is the softest, without much strength, but running very fluid, and good for mixing with harder varieties of pig or with scrap iron. No. 2 is harder and stronger, with a closer grain and a brighter surface. No. 3 is yet harder and stronger, with a metallic lustre. No. 4 is termed "Bright;" No. 5 "Mottled;" and No. 6 "White:" the latter being very hard, brittle, and unfit for use alone. As the color and structure of cast iron depends much upon the length of time in which it cools from a fluid state, small castings and the surfaces of large ones, are almost always white and hard. The various ores and the mineral fuels used in smelting frequently contain substances which injure the quality of the iron, sulphur and phosphorus being among the worst. The former renders the iron, when made into wrought bars, fusible, and hard to weld, being brittle when hot, or "hot short." The latter makes the iron brittle when cold, or "cold short." The hot blast, while saving fuel, and producing a larger yield, also causes the iron to combine with a larger quantity of impurities. Iron for purposes requiring great strength should be made only from carefully selected ores, reduced with charcoal, and the cold blast.

For the strength of the various sorts of cast iron, we are indebted chiefly to the investigations of Mr. Hodgkinson.* He

* Experimental Researches on the Strength and other Properties of Cast Iron; and Report of the Commissioners appointed to inquire into the Application of Iron to Railway Structures.

found the mean ultimate tensile strength of 27 various irons to be 15,679 pounds per square inch. The varieties experimented upon were Nos. 1, 2, and 3, both hot and cold blast, of the best Scottish and Welch manufacture. The addition of a certain percentage of malleable scrap, first proposed by Mr. Morries Sterling, producing what is termed toughened cast iron, increases very much the strength. Mr. Hodgkinson found the tensile strength of Calder (Lanarkshire) No. 1, hot blast, mixed with 20 per cent. of malleable scrap, to be 25,764 pounds per square inch. The strength of Staffordshire No. 1, hot blast, with 15 per cent. of scrap, was 23,461 pounds.

As a general thing, the cold blast irons are superior in strength to the hot blast, although much depends upon the original character of the metal.*

Remelting, or prolonged fusion, increases the density and strength of cast iron, though these processes involve a waste of material. Experiments by Mr. Fairbairn, in which a ton of No. 3 hot blast iron was melted 18 times in succession, showed the strength to increase up to the 12th melting; after which it decreased. Experiments by Major Wade,† of the United States Army, gave the results following, with bars of No. 1 Greenwood pig iron:—

	Density.	Tensile Strength in Pounds per Square Inch.
Crude pig iron,	7,032	15,129
Remelted once,	7,086	21,344
Remelted twice,	7,198	30,107
Remelted three times,	7,301	35,786

* Professor Rankine observes (Civil Engineering, p. 500) that the experiments of Fairbairn and Hodgkinson indicate that with the same kind of ore and fuel, No. 1 cold blast is in general superior to No. 1 hot blast; that No. 2 hot and cold are about equally good; that No. 3 hot blast is in general superior to No. 3 cold blast; and that the average quality of the iron, on the whole, is nearly the same with the hot as with the cold blast.

† The important experiments of Major Wade are to be found in the Report on the Strength and other Properties of Metals for Cannon. By officers of the Ordnance Department, United States Army. Philadelphia, 1856.

The effect of remelting varies with the quality of the iron. The soft, gray varieties will of course gain more by decarbonizing than the harder irons. Another mode of refining the iron is by keeping it in a state of fusion for a long time, by which carbon is removed, and the tenacity increased. The following experiments of Major Wade, upon Stockbridge iron, show the effect of prolonged fusion: —

	Density.	Tensile strength in Pounds per Square Inch.
Iron in fusion ½ hour, ...	7,187 . .	17,843
Iron in fusion 1 hour, ...	7,217 . .	20,127
Iron in fusion 1½ hours, . .	7,250 . .	24,387
Iron in fusion 2 hours, . .	7,279 . .	34,496

Both remelting and prolonged fusion, however, may be carried too far: as the carbon is removed, the iron becomes less fluid, fills the moulds less perfectly, and produces too hard and brittle a metal.

Cast iron, being liable to hidden defects from unequal contraction in cooling, and other causes, and having a low, tensile strength, is not employed to resist a direct tension. It is, however, subject to such a strain in the lower flanges of girders, and thus the preceding figures will be of use.

It is when employed to resist compressive forces that we obtain the full value of cast iron, as its resistance to crushing is very great. The mean compressive strength of 16 several irons, tested by Mr. Hodgkinson, was 86,284 pounds per square inch. The resistance of toughened iron made of Calder (Lanarkshire) No. 1 hot blast, with 20 per cent. of malleable scrap, was 122,395 pounds, and Staffordshire No. 1 hot blast, with 15 per cent. of scrap, 144,264 pounds.

Wrought or malleable iron *should be* pure iron. It, however, generally contains ½ per cent. or less of carbon, and except the best varieties other impurities. Cast iron contains from 2 to 5 per cent. of carbon, and its conversion to wrought iron consists in the removal of the excess of carbon, which is effected by melting the

pig iron, and keeping it in a state of fusion in contact with air in a reverberatory or puddling furnace, until its carbon by combining with the proper quantity of oxygen is nearly or quite removed, when the iron loses its fluidity, and becoming stiff, is worked up into a ball, which is placed under a hammer to be shingled, or into a squeezer to be compressed, in order to work out the liquid cinder. The oblong mass thus produced, termed a "bloom," is then raised to a welding temperature, in a reheating furnace, and again hammered, or if it retains heat enough, is at once passed through the roughing rolls, by which it becomes a flat bar. The bars thus made are cut into lengths and piled, or "fagotted" into bundles. Finally, these piles are again heated, and passed through the finishing rolls, by which they become round, square, flat, angle, or other bars. In some cases, when a superior quality of iron is desired, the pig iron is refined before being puddled as above. This process, by which a part of the carbon is removed, consists in remelting the pig in a refining furnace, keeping it for some time in a state of fusion in contact with air, and afterwards running it into large flat cast iron moulds, thus forming large plates of hard and brittle iron, which are broken up and worked in the puddling furnace, as before described. The process of cutting and piling the rough bars from the puddle rolls, is, in some cases, repeated, in order to get a more thoroughly worked and superior iron. Reworking, however, can be carried too far. Good wrought iron, compact and free from impurities, after two or three workings, reaches its best condition, after which additional heating injures it.

Wrought iron varies in quality according to the original ore and the thoroughness of the treament during the manufacture. A bar, to be strong and tough, should have a fracture of a clear blue-gray, and a uniform, fine, close, granular structure. It should bear a high welding heat, and bend in any direction when hot or cold, without breaking. Cold short iron has a lighter colored fracture, and, though tough when heated, is very brittle when cold. Hot short iron is dark, and without lus-

tre, and, while tough and useful for many purposes when cold, is brittle and unmanageable when hot. A bar of pure, tough iron will be clean and smooth, free from cracks along the edges, and, when drawn out into small bars, the surface of fracture will show the grains drawn out into fibres with a silky lustre.

The most extensive, and by far the most complete and valuable series of experiments upon the tensile strength of wrought iron, are those made by Mr. Kirkaldy, at the works of Messrs. Robert Napier & Sons, of Glasgow.* The following is a general summary of these experiments:—

	Tensile Strength in Pounds per Square Inch.		
	Highest.	Lowest.	Mean.
188 Bars, rolled,	68,848	44,584	57,555
72 Angle Iron, &c.,	63,715	37,909	54,729
167 Plates, lengthwise,	62,544	37,474	50,737
160 Plates, crosswise,	60,756	32,450	46,171

The mean of 120 experiments upon double refined iron, made by H. Burden & Co., Troy, N. Y., for the lower chord links, and the main ties of the bridge across the Mississippi at Quincy, gave as the mean ultimate tensile strength 63,783 pounds per square inch; the minimum being 50,773, and the maximum 80,023 pounds.

The ultimate resistance of wrought iron to crushing is from 30,000 to 40,000 pounds per square inch, and thus much less than that of cast iron; but the compression of wrought iron under the same weight is, for a considerable range, only about half that of cast iron, as shown by the following results obtained by Mr.

* Results of an Experimental Inquiry into the Tensile Strength and other Properties of various Kinds of Wrought Iron and Steel. By David Kirkaldy. Second edition. London, 1866. The "Concluding Observations" of the above work, to which the reader's attention is particularly directed, are given in the Appendix. The several irons tested by Mr. Kirkaldy were not selected as being of extra quality, but were obtained from the various brands commonly for sale in the market. When iron is prepared for a purpose requiring special strength and safety, as in bridge work, it will reach the numbers given in the first column of the above table.

Hodgkinson, with bars one inch square and ten feet long, of Low Moor cast and wrought iron.

Weight applied in Pounds.	Decrease of Length in Decimals of an Inch	
	Wrought Iron.	Cast Iron.
5,098	.028	.048
9,578	.052	.091
14,058	.073	.137
20,778	.107	.208
25,258	.130	.254
29,738	.154	.305

The shearing strength of wrought iron, or its resistance to being cut across, is about the same as its tensile strength, and is proportional to its sectional area. Experiments upon rods of $\frac{7}{8}$ inch rivet iron of excellent quality (area, 0.6013 square inch), gave for single shear (mean of four trials) 24.15 tons per square inch of section as the resistance to shearing. The mean of eight experiments with the same rod in double shear gave 23.3 tons per square inch of section, the ultimate tensile strength being 24 tons. Two plates $\frac{5}{8}$ inch thick were riveted together by a single $\frac{7}{8}$ inch rivet, which was sheared by 12,267 pounds, or 20.4 tons per inch. Three plates being joined by a similar rivet, the latter was cut in two places, by the middle plate, by a weight of 26.8 tons, or 22.3 tons per square inch of section.*

The Nature and Strength of Steel.

The discovery of new modes of producing steel has brought this material into use for engineering purposes of late years, to an extent before unknown; and its application to railway and bridge work is becoming every day more and more important. The processes of manufacture are being rapidly improved, and

* Clark's Britannia and Conway Bridges. Vol. I. pp. 390–392. Mr. Rankine gives 27.700 pounds per square inch as the resistance of cast iron to shearing, and 50.000 pounds as the resistance of wrought iron to the same strain.

its cost reduced to such an extent, that, considering its great strength and durability, it must, for many purposes, soon be preferable to iron. Steel is simply iron, with from $\frac{1}{2}$ to $1\frac{1}{2}$ per cent. of carbon, and is made either by the old process of adding carbon to malleable iron, or by the new mode of abstracting carbon from cast iron. The old method of heating iron in the presence of carbon, termed cementation, and the subsequent operations of shearing, piling, hammering, and rolling, or of cutting up and melting the cemented bars to make cast steel, are employed for the smaller work, such as tools: for rails, plates, and bridge work, the new methods are used. The invention of Mr. Bessemer, by which so great a change has been made in the iron and steel trade, consists in forcing air through molten pig iron until the requisite decarbonization is effected, or else until the carbon and other impurities are almost entirely removed, and adding to the molten and nearly pure iron thus left carbon and manganese in the proper proportion, when the mass is run into ingots, and hammered and rolled in the same way as the blooms referred to in the preceding remarks upon iron, making thus homogeneous cast steel. Another process consists in puddling pig iron, and stopping the operation when the proper degree of decarbonization is reached, thus forming puddled steel. Between steel and iron is the so-termed semi-steel, containing less than $\frac{1}{2}$ per cent. of carbon, and having qualities intermediate between iron and steel. As a general thing, the greater the proportion of carbon, the more fusible is the metal, and the harder and stronger, but the less adapted for forging and welding.*

The following figures are from Mr. Kirkaldy's work, before referred to: —

* Numerous valuable papers upon the important subject of the steel manufacture, selected from the best European publications, may be found in Van Nostrand's Magazine for 1869 and 1870.

TENSILE STRENGTH OF STEEL BARS.

Manufacturer.	Breaking Weight in Pounds per Square Inch. Original Area.	Fractured Area.
Krupp's cast steel for bolts, *rolled*,	92,015	139,434
Mersey Co.'s puddled steel, *forged*,	71,486	110,451
Blochairn puddled steel, *rolled bars*,	70,166	84,871
Blochairn puddled, *forged from slabs*,	62,255	80,370
Blochairn puddled, *forged from rolled bars*,	62,769	71,231
Bars forged from cast steel, for tools,	101,151	142,070
Bars forged from cast steel, for tools,	132,909	139,124

The ratio between the first and second columns shows the character of the metal, whether hard or tough.

The tensile strength of steel plates, from the same authority, is as follows: —

TENSILE STRENGTH OF STEEL PLATES.

Name of Manufacturer.	Breaking Weight in Pounds per Square Inch. Original Area.	Fractured Area.
Turton & Co., cast steel, $\frac{1}{4}$ inch thick.		
lengthwise,	94,289	100,063
crosswise,	96,308	111,811
Shortridge & Co., cast steel, $\frac{3}{16}$ inch thick.		
lengthwise,	96,280	114,106
crosswise,	97,150	114,300
Naylor, Vickers, & Co., cast steel, $\frac{1}{4}$ inch thick.		
lengthwise,	81,719	104,232
crosswise,	87,150	112,018
Mersey Co., puddled steel, $\frac{1}{8}$ and $\frac{3}{16}$.		
lengthwise,	101,450	109,552
crosswise,	84,968	91,746
Blochairn puddled steel, $\frac{3}{16}$ inch.		
lengthwise,	102,234	108,879
crosswise,	84,398	87,877

From the experiments of Mr. Fairbairn we have the following (Report of British Association for 1867):—

TENSILE STRENGTH OF VARIOUS KINDS OF STEEL.

Manufacturers.	Tensile Strength per Square Inch in Pounds.	Elongation in per cent. of the Original Length.
C. Cammel & Co.		
Hard Bessemer steel,	89,121	20.87
Soft Bessemer steel,	81,483	20.43
Naylor & Vickers.		
"Axle steel," cast,	88,665	16.25
"Tyre steel," cast,	91,520	9.00
Henry Bessemer.		
Hard steel,	103,085	1.87
Milder steel,	88,175	20.00
Soft steel,	78,606	19.12
S. Osborne.		
Toughened cast steel for shafts,	103,116	5.25
Cast steel for boiler plates,	111,676	13.50
Best cast steel for taps and dies,	98,790	1.68
Extra best tool cast steel,	85,724	0.43

But few experiments have been made to determine the crushing strength of steel. Major Wade gives for hardened cast steel, for tools, from 350,000 to 390,000 pounds per square inch, in round numbers, for the ultimate resistance to compression.* Professor Rankine gives 269,000 pounds as the crushing load for small blocks of cast steel.† Mr. Fairbairn states that 225,568 pounds, or 100.7 tons, produced no sensible fracture upon 32 several bars, while 23 other bars were more or less broken. All of them, however, were compressed and spread out laterally to a considerable

* Report on materials for cannon, by officers of Ordnance Department, U. S. A.

† Civil Engineering, p. 587.

extent.* "The crushing strength of steel," says Mr. Stoney, "is so high, that 12 or 15 tons per square inch is perhaps a safe compressive working strain, when the material is not allowed to deflect. Experiments are, however, still wanting to determine the safe working compression of steel, when subject to flexure in such sections as those usual in pillars and the struts of girders. Till such are made, it will probably be safe to adopt one and a half times the strain that a similar section of wrought iron would safely carry." †

The shearing strength of rivet steel was found by Mr. Kirkaldy, from a mean of 17 experiments, to be 26.2 per cent. less than the tensile resistance; the former being 63,796 pounds, and the latter 86,450 pounds per square inch; or the shearing strength may be taken as three fourths of the tensile strength. ‡

In a steel bridge made for the Herljunga and Wernersborg Railway, in Sweden, the span being $137\frac{1}{2}$ feet, the parts are calculated to bear a working strain of 8 tons per square inch; every piece being tested to double that amount before going into the bridge. § The specification for the (crucible) steel for the St. Louis Bridge calls for a compressive strain of 60,000 pounds, and a tensile strain of 40,000 pounds per inch without permanent set, and a tensile strain of 100,000 pounds without fracture, for the steel tubes, which are to act by compression. ||

Mr. Fairbairn tested a plate girder made of a tough variety of the Barrow Hæmatite steel, the span (between supports) being 13.9 feet, the height 12 inches, the web $\frac{1}{4}$ inch thick, and the flanges made of two angle-irons, each $\frac{1}{4} \times 2\frac{1}{2}$ inches, riveted back to back to the web. The girder broke through the lower flange with 18.1 tons; the strength of a wrought iron beam of the same size being 11.8 tons. Mr. Fairbairn observes, "Taking,

* Report of British Association for Advancement of Science, 1867.

† Stoney, Theory of Strains, Vol. II. p. 390.

‡ Experiments on Wrought Iron and Steel.

§ London Engineer, Vol. XXII. p. 240. 1866.

|| Report of Engineer of Illinois and St. Louis Bridge, 1870.

however, the strength of steel beams, and the amount of deflection, when submitted to a transverse strain, it will be found that they are not only one third stronger than those of the best wrought iron, but that they are much superior in their powers to resist impact, and therefore more secure under the influence of a rolling load, or severe vibratory action long repeated." *

Steel plates, as well as those of hard iron, are injured by punching, and the harder and more brittle the metal, the greater the damage. Punching, as compared with drilling, was found to reduce the strength of $\frac{5}{16}$ inch steel plates 33 per cent.

Steel bars heated and plunged into oil are increased in strength and toughness to a remarkable extent. Some of the hard steels, highly heated, are shown by Mr. Kirkaldy's experiments to have gained nearly 80 per cent. Plates were found to gain from 12 to 56 per cent., and were not only hardened, but toughened. Plates joined by rivets and hardened in oil are stated by Mr. Kirkaldy to be fully equal in strength to unjointed soft plates; or, in other words, the loss by riveting is more than counterbalanced by the increase in strength by hardening.

* Experimental Inquiry on Barrow Hæmatite Steel. London, 1869.

CHAPTER VIII.

STRENGTH OF MATERIALS. — RULES FOR PRACTICE.

Factors of Safety.

The figures given in the preceding chapter show the ultimate or breaking strains of the several materials. Only a small part of this, however, is to be used in practice. The ratio of the breaking to the working strain has been variously fixed by different engineers for the several materials, and for the various conditions under which the load is applied. A bar or beam may be loaded with a greater weight applied as a permanent or dead load than would be safe as a moving or rolling weight. A load may be brought upon any material in an easy and gradual manner so as not to damage it, while the same load could not be suddenly and violently applied without injury. The margin for safety should be greater with a material liable to contain hidden defects than with one which is not so. It should be greater for any member of a structure where it is subjected to several different kinds of strain, than when subjected only to a single form of strain. The rule in structures, having so important an office to perform as railway bridges, should in all cases be *Absolute Safety under all conditions*. It is common to make the working strain some factor of the breaking load; and this is, perhaps, the simplest mode of proceeding in the present state of our knowledge of materials.*

* It has been proposed to make the working strain such that the extension or compression of the material shall not exceed a certain part of that corre-

The British Board of Trade fixes the greatest strain that shall come upon the material in a wrought iron bridge, from the combined weight of the bridge and load, at 5 tons per square inch. The ordinary practice in England allows 5 tons per square inch of the *net* section for wrought iron, subject to tension, and 4 tons per inch of the *gross* section when subject to compression. This is about one fourth of the ultimate strength, and, when care is taken to use only the best material and workmanship, is perfectly safe and reliable. The practice of the French engineers allows 3.8 tons per square inch upon the gross section of both compression and extension members of wrought iron bridges; the loss by the rivet-holes in the tension members being assumed equivalent to the difference between the tensile and compressive strength of the material.

In the Louisville Bridge, Mr. Fink has adopted 7 as a factor of safety for the cast iron chords, under an extreme load, and from 5 to 6 for the posts and braces. The wrought iron work in the same bridge is strained from 7,000 to 12,000 pounds per inch, by the maximum load, according to its position in the structure. In the Mississippi Bridge, at Quincy, the lower chords and ties are strained from $\frac{1}{7}$ to $\frac{1}{8}$ of the ultimate strength by the greatest load. The cables of the Niagara Suspension Bridge have an ultimate strength of five times the utmost probable load. In the fine wrought iron elliptical arch, upon the Pennsylvania Railroad, at Thirtieth Street, Philadelphia, Mr. Wilson has employed a factor of safety of 6 throughout. In the Canestota Bridge, upon the New York Central Railway, a wrought iron lattice, Mr. Hilton has adopted 5 tons per inch for the working strain upon the net section of the lower chords, 4 tons for the compression upon the gross section of the top chords, 5 tons upon the tension bars of the lattice, and 3 tons upon the compression bars. In the bridge across the Ohio, at Steubenville, of which the channel

sponding to the limit of elasticity; but this method is hardly definite enough for practice, as we have so little knowledge of this limit in various materials — if, indeed, there is any precise limit.

span is 320 feet, a rolling load of 3000 pounds per foot produces 10 000 pounds of tension on the lower chords, and the same amount of compression upon the top chords, the latter being of cast iron. The factor of safety for the braces is $\frac{1}{8}$.

In the bridge across the Connecticut, at Warehouse Point, on the Hartford and New Haven Railroad, the maximum tensile strain is five tons, the compression on the top chords four tons, and on the posts three tons, all the members being of wrought iron. The channel span is 177 feet, and is proportioned for a load of $2\frac{1}{2}$ tons per foot, including the weight of the bridge.

In the Kansas City Bridge, across the Missouri, the central tie rods and the truss rods of the floor beams, were proportioned for 10,000 pounds per inch; the end ties and lower chords 12,000 pounds: the compression on the top chords, which are of wood, 800 pounds per inch; and for the wooden braces a factor of 7 was adopted.

In the valuable pamphlet by Mr. C. Shaler Smith,* comparing the several principal bridge trusses now in use, he observes, "In determining the proportion of safe to breaking strain, respect should be had to the frequency with which the parts are subjected to stress by the moving load, as this will manifestly influence their powers of endurance. The primary system of a truss, that is, the system which upholds the entire structure, when weighted with its maximum load from end to end, such as the chords of a Murphy, or Triangular, or the main and secondary tension bars of a Fink bridge, may be safely subjected to a heavier strain than the tertiary and panel systems of the Fink, or the panel systems of the other trusses, the latter of which are fully strained by the engine leading every train; while to strain the primary systems to the utmost limit allowed would require a train made wholly of engines." † In making comparisons of several forms of truss, Mr.

* Comparative Analysis of the Fink, Murphy, Bollman, and Triangular Trusses. By C. Shaler Smith, C. E. Baltimore, 1865.

† This idea originated, we believe, with Mr. Fink.

Smith accordingly uses 10,000 pounds per inch for the quaternary system of the Fink bridge, and for the middle panels of the other trusses, as the working load for the wrought iron tension bars. For the tertiary system of the Fink, and the first four panels of the other trusses, 11,000 pounds is adopted, and for the primary and secondary systems of the Fink, and the chords of the Murphy and Triangular, 12,000 pounds. For the cast iron parts, for chords and large posts when the metal is an inch and over in thickness, and the length not over fifteen diameters, the factor is taken at $\frac{1}{5}$; with metal not less than $\frac{5}{8}$, and length not over twenty-five diameters, $\frac{1}{7}$; and with metal less than $\frac{5}{8}$, and length over twenty-five diameters, from $\frac{1}{10}$ to $\frac{1}{40}$.

Mr. Stoney (Theory of Strains, Vol. II. p. 387) gives the following figures. The working tension in the lower flanges (chords) of the Conway Tubular Bridge, span 400 feet, from the permanent weight of the structure, is 4.385 tons per square inch of gross section: the compression in the upper flanges is 3.063 tons per inch. The working tension, with one ton per foot of rolling load, is 6.85 tons, and the compression 5.03 tons. In obtaining the above figures, the vertical webs are not taken into account, and the rivet holes are not deducted from the flanges. In the Newark Dike Bridge, $240\frac{1}{2}$ feet span, the top chords and braces of cast iron, under a rolling load of a ton per foot, are strained to 5 tons per inch, and the lower chords and ties of wrought iron are strained to the same amount. In the Boyne Viaduct, a wrought iron lattice, 264 feet span, the working tensile strain for the net area of the lower chords, under a rolling load of one ton per foot, is 5 tons per inch, and the compression on top chords 4.5 tons. In the Charing Cross Bridge, a lattice of 154 feet span, with four lines of railway, the working tensile strain at $1\frac{1}{4}$ tons per lineal foot on each of the lines, is 5 tons per inch, and the compressive strain 4 tons.

In the Crumlin Viaduct, a Warren girder, 148 feet span, the maximum tensile strain in the diagonals from the bridge and load is 6.65 tons per square inch of the net section; the tension on

the lower chords from the same strain is 5.75 tons per square inch of net section; and the maximum compression on the top chords is 4.31 tons per inch of *gross* section.

Mr. Unwin (Iron Bridges and Roofs) gives the following strains in several recent wrought iron bridges, the tension being for the net section, and the compression for the gross section of the metal.

Name of Bridge.	Tension, Tons per Inch.	Compression, Tons per Inch.
Passau, Lattice,	$5\frac{1}{2}$ to 6	$4\frac{1}{4}$ to $5\frac{1}{4}$
Penrith, Tubular Girder,	$4\frac{3}{4}$	$4\frac{1}{4}$
Place de l'Europe, Lattice,	$4\frac{1}{4}$	$3\frac{3}{4}$
Lough Ken, Bowstring,	4	$3\frac{3}{8}$
Isère, Lattice,	$4\frac{1}{2}$	$3\frac{1}{2}$

The following figures show the working strains upon several of the larger suspension bridges:—

Name of Bridge.	Length of the Chord of the Catenary in Feet.	Working Strain in Tons per Inch.		Factor of Safety.
		From the Bridge.	From Bridge and Load.	
Menai,	580	4.21	8.00	3.9
Hammersmith,	$422\frac{1}{4}$	5.38	9.36	3.3
Pesth,	666	5.01	8.11	3.9
Chelsea,	348	4.36	8.07	3.9
Clifton,	$702\frac{1}{4}$	2.90	5.03	6.2
Niagara,	821	6.70	8.40	5.0

The load from which column 3 is computed is assumed at 80 pounds per square foot of the floor, except in the case of the Niagara Bridge, where it is reckoned at 250 tons in all. The factors in the last column are obtained by assuming the ultimate tensile strength of the chain links, which are made of very superior iron, at 70,000 pounds per inch; and for the Niagara Bridge, which has wire cables, at 100,000 pounds.

Mr. Humber, in his large work on bridges, furnishes the following concerning several cast iron arches:—

Name of Bridge.	Length of Span. ′ ″	Versed Sine of Arch. ′ ″	Strain per Sq Inch, in Tons.	Depth of Rib. ′ ″
Austerlitz,	106 0	10 7	2.78	4 4
Carrousel,	154 2	16 1	1.46	2 10
St. Denis,	102 5	11 4	1.37	2 10
Nevers,	137 9	15 0	1.90	3 9
Rhone,	197 10	16 5	2.37	5 7
Westminster,	120 0	20 0	3.00	

In determining the factor of safety, regard should be had to the proportion that exists between the dead and the live load. A rolling load has been assumed by engineers (and for practical purposes the assumption is probably correct) as equivalent to a dead load of double the amount. In determining the factor of safety, therefore, we may reduce the whole load to an equivalent dead weight, which will be equal to twice the actual live load added to the actual dead load ; or we may, as Mr. Rankine suggests, multiply each part of the load by its proper factor of safety, and add together the products, the sum being the breaking load to which the structure is to be adapted.* Thus, regarding a ton of live load as equivalent to two tons of dead load, and adopting a factor of $\frac{1}{6}$ for live and $\frac{1}{3}$ for dead weights, if we have 200,000 pounds of dead load, and 100,000 of live load, the equivalent dead weight is 400,000 ; and the factor being 3, the breaking weight would be 1,200,000 pounds ; or we may multiply the 200,000 by 3, and the 100,000 by 6, giving the same result. If the dead load was 100,000, and the live load 200,000, we should find the breaking weight to be 1,500,000 pounds, in place of the 1,200,000 above.

Mr. Fairbairn subjected a wrought iron plate girder, of 20 feet span and 16 inches deep, to repeated deflections by means of a loaded lever, with the following results : A load of 2.96 tons ap-

* Rankine. Civil Engineering, p. 222. Unwin, Iron Bridges, etc., p. 37. Stoney, Vol. II. Chap. XXVI.

plied at the centre at the rate of about eight changes a minute for 596,790 times, producing a deflection of 0.17 inch, and straining the net section of the bottom flange 5.92 tons per inch, produced no apparent change. Increasing the load to 3.5 tons, 403,210 additional deflections of 0.23 inch, straining the bottom flange to 7 tons per inch produced no visible change. Increasing the load to 4.68 tons, 5175 additional deflections, depressing the girder 0.35 inch, and straining the lower flange 9.36 tons per inch, produced a permanent set of 0.05 inch, and tore the lower flange apart near the middle. The girder being repaired, a load of 2.96 tons, applied 3,124,000 times, produced no permanent set. A load of 4 tons, applied 313,000 times, produced a deflection of 0.20 inch, straining the lower flange to 8 tons per inch, and broke the girder close to the splice over the former fracture. Thus a strain of 5.92 tons per inch of the lower flange section, the breaking weight being 20 tons per inch, would seem to be a safe load.

Mr. Fairbairn found also that small cast iron posts 6 feet long, and an inch in diameter, bore a constant load of half the breaking weight for three years without changing; while with three fourths of the breaking load the deflection was slightly increasing at the end of the same time. He also subjected model bars of cast iron 4′ 6″ long between the supports, and an inch square, to a cross strain of from one half to over three fourths of the breaking weight. These loads were borne with a slightly increasing deflection for over five years. The greater part of the deflection occurred during the first year, after which it remained very nearly constant. In the report of the commissioners upon the application of iron to railway structures, experiments are referred to in which cast iron bars, supported at the ends, were subjected to successive deflections, — in some cases as many as 100,000, — by means of a cam, at the rate of four per minute. When the depressions were $\frac{1}{3}$ of the ultimate deflection, the bars were apparently not weakened. When, however, the depressions were $\frac{1}{2}$ of the ultimate deflection, the bars were broken with less than

900 flexures. A load equal to half the breaking weight, moved backwards and forwards from one end to the other of a cast iron bar, produced no apparent weakness by 96,000 transits.

For cast iron bridges the British Board of Trade has directed that the breaking weight should not be less than three times the permanent load from the bridge itself, added to six times the greatest rolling load. The general opinion of engineers, however, is, that the breaking weight should be four times the total load for good sized castings supporting a dead weight, six times for large castings subjected to changes of load and vibrations, as in railway bridges, and eight times for small castings and for work subjected to particularly violent and sudden shocks.

"The resisting powers of beams," says Mr. Fairbairn, "of whatever material they may be composed, are like the muscles of the animal frame when strained beyond their reasonable powers of resistance. They may for a time endure the load, and probably a few repetitions of it ; but the result generally is, either the rupture of the several parts, or the total suspension of those qualities by which its elasticity and powers of restoration are maintained. It therefore follows that every description of material, when subjected to a transverse strain, should never be urged to greater endurance than may be required to straighten the fibres, or arrange the molecules of its crystalline structure. Any strain beyond that point is attended with risk ; and in every case where the beam is subject to alternate change of vibration, to dead weight, and the force of impact, it is safer to multiply the load by 4, than by 3, as the ultimate strength of the beam. In girders for railway bridges the multiplier should never be less than 5 ; and in most cases even 6 is preferable, owing to the great weight and high velocities with which trains pass over a continuous line of rail, involving equally severe tests of impactive force on every structure, whether beams or bridges, that have to support the immense weight of railway traffic, varying in speed from 25 to 50 miles an hour." *

* Application of Cast and Wrought Iron to Building Purposes, p. 153.

Practical Rules for the Use of Wood.

As shown upon a preceding page, the ultimate tensile strength of oak and pine ranges from 10,000 to 15,000 pounds per inch, and the compressive strength from 5000 to 7000; there being in general little difference in regard to these strains between pine and oak. Owing to the lack of seasoning, and to numerous other causes affecting the timber used in construction, it has not been considered proper to employ more than one eighth or one tenth of the ultimate strength, or from 800 to 1000 pounds per square inch of the net section for tension, when applied to railway bridge work. For simple dead weight double this may be employed; and for temporary works even more. For compression, we may adopt from 600 to 800 pounds per square inch as the safe working load; but this may be applied to the gross area of section, and not alone to the net area, as in the case of tension, as a compressed joint, where the work is good and the bearing perfect, reduces the available resistance little or none.*

The shearing of timber in the direction of the grain, as already stated, is for pine, spruce, and fir, from 600 to 800 pounds, and for oak 2300 pounds. For well seasoned pine we may use from 100 to 125 pounds per inch to resist this strain; for oak 400 pounds may be employed. The resistance to crushing across the fibres, as where the braces or posts in a truss thrust against the chord, may be taken at 300 pounds per inch for pine, and 500 pounds for oak. In such cases it is well to enlarge the bearings, when the pressure is extreme, by flanged castings, or by hard wood blocks, in order to distribute the strain; and better still, to apply the braces as in the Howe truss, so as to avoid entirely crushing the chord.

* Mr. Whipple adopts 1000 pounds as the safe rule for tension, and from 100 to 1000 pounds for compression, according to the ratio of the length to the diameter, as shown in advance. Mr. Haupt adopts 1000 pounds both for tension and compression, but observes that 800 would probably be more nearly the true medium between safety and economy. — Whipple, *Bridge Building*, p. 94; Haupt, *Bridge Construction*, p. 165.

The ultimate shearing strength of white oak treenails has already been given as 4000 pounds per square inch of the area. Of this, 600 pounds may safely be employed in practice, supposing the pin to fail by direct shearing. If, however, the pins are so used as to be subjected to a transverse strain, i. e., if employed to connect timbers not placed closely together, the strength is to be found by the rule given in advance for transverse strains.

The Strength of Posts or Columns.

In determining the proper dimensions of posts, columns, braces, or other compression members of a structure, we have to consider not only their ability to resist crushing, but also, and more particularly, their power of resisting flexure, as it is generally in this way that they fail. Mr. Hodgkinson divides pillars into three classes. First, short pillars, the length of which is less than 4 or 5 diameters, which fail by simple crushing. This crushing may take either of several forms, according to the nature of the material. Crushing by splitting into prisms, or pyramids, occurs in granular, crystalline, and vitreous materials, such as glass, stone, and cast iron. Crushing by bulging, or by spreading out laterally under pressure, takes place in ductile materials, as lead, wrought iron, or copper. Crushing by buckling, puckering, crumpling, or wrinkling, commonly takes place in thin plates of malleable material. The second class, or medium pillars, which yield partly by crushing and partly by flexure, have a length of over 5 and less than 30 diameters, if of cast iron or timber, and under 60 diameters if of wrought iron. The third class, or long pillars, which fail altogether by flexure, are over 30 diameters if of cast iron or wood, and 60 diameters if of wrought iron. These remarks refer to columns whose ends are truly flat and firmly bedded. A column with the ends rough from the foundery, so that the load is applied to a few detached points, or with rounded ends, so that the load is applied only to the centre line, has only one third of the strength of a flat-ended pillar

of the same length, for columns of the third class, and from one third to two thirds of the strength for columns of the second class. The strength of a post with one end flat and the other round is the mean between the strength of pillars of the same dimensions, one having both ends flat and the other both ends round. Discs upon the ends of posts add very little to the strength, but they are of use in forming connections. Enlarging the diameter of posts in the middle increases their strength in certain cases, but not to any great degree. Mr. Hodgkinson found the strength of solid cast iron cylindrical pillars, with rounded ends, to be increased by about one seventh by enlarging the middle. Pillars with disc ends but little enlarged were no stronger; but when the diameter was enlarged fifty per cent. they were from one eighth to one ninth stronger. Hollow cast iron pillars were not found to be strengthened at all by enlarging.

In casting hollow pillars one side often comes out thicker than the other; but as the thinner side is commonly harder than the thicker one, the strength of the two does not vary much, provided the difference in the two sides is not over about a fourth of the average thickness of the metal.* Thin castings being harder than thick ones, we not only by making a cast iron pillar hollow remove the material from the axis, thus enabling it the better to resist flexure, but we get, too, a greater unit of crushing strength. Mr. Hodgkinson's experiments upon various forms of section with rounded ends gave the following results: **+** 17,578 **H** 29,571, **O** 39,645. The strength of solid, round, square, and triangular cast iron columns of the same weight and length were found to be as 100, 93, and 110, respectively. It was also

* If a cast iron column is taken from the mould too quick, it is apt to be bent in the handling. If one side is much thicker than the other, the former cools the last, and often brings a severe strain upon the thin side, and frequently bends the pillar. The thickness of hollow cast iron columns, when cast upon the side, as they usually are, may vary from the displacement of the core — a fact that may be detected either by drilling a small hole in the side of the pillar, or by a careful application of callipers.

found that round tubular pillars were, in general, stronger than rectangular ones; the crushing unit strain of the latter being greater the thicker the plates were in proportion to the width of the tube. The strongest form for a rectangular pillar to resist buckling, according to Mr. Stoney, is one in the angles of which the chief part of the material is concentrated, making the sides, thin plates, or lattice work, merely sufficient to withstand flexure of the angles; in which case the thin plates act the part of the web, and the angles act as the flanges of a girder.* According to Mr. Hodgkinson's experiments the relative strength of long pillars of different materials is as follows:—

Cast Steel, not hardened, ...	2518
Wrought Iron, best Staffordshire, ...	1745
Cast Iron, Low Moor, ...	1000
Dantzic Oak, ...	109
Red Deal, ...	78

The position of fracture, in long, uniform cast iron columns, depends upon the form of the ends. When both ends are rounded, the pillar sustains a single flexure, and breaks in the middle. With one end flat, and the other rounded, the column sustains a double flexure, and breaks at about one third the length from the rounded end. With both ends flat, there is a triple flexure, and the column breaks at the middle and near each end.†

Numerous formulæ for obtaining the strength of columns of different materials, and many tables giving the weight that may be safely borne by solid and hollow cylindrical cast iron pillars,

* Theory of Strains. Vol. II. p. 211, 220–223; Appendix, p. 463–467, and Plate V.

† "A wrought iron pillar," says Mr. Stoney, Vol. II. p. 189, "may be expected to fail on the concave side, as its power to resist compression is less than that to resist extension. A long pillar of cast iron, on the contrary, will probably fail by the convex side tearing asunder, as the compressive strength of cast iron greatly exceeds its tenacity."

have been published. Owing however to the difference in the rules, and more particularly to the difference in the factor of safety assumed by the several authorities, the results given are very far from being alike, and in many cases are quite unsatisfactory.*

The tables prepared by Mr. Francis are among the best. They refer to solid and hollow cast iron pillars, with round ends, but by the accompanying rules they apply also to wrought iron and wood, and to other sections than the circle. The factor of safety adopted is *five*. The tables give the safe load for columns with round or with unfinished ends. For pillars with ends turned flat and put up with ordinary care, 50 per cent. is added to the tabular number. For a square pillar, with a side equal to the tabulated diameter, 50 per cent. is added to the load for a cylinder. A column of oak is taken as having $\frac{1088}{1000}$ of the strength of a similar cast iron one, and a column of pine as having $\frac{785}{1000}$ of the same. A wrought iron pillar, not shorter than 60 diameters, is considered as having $\frac{1745}{1000}$ of the strength of a cast iron one of the same dimensions. Compared with other results, those of Mr. Francis are quite low, and are certainly safe.

Mr. Whipple, in his work upon Bridges, gives a table of the safe weight, in pounds per inch, that may be placed upon cast and wrought iron columns of different forms, the load varying according to the ratio between the length and diameter of the

* The tables given in the numerous pocket books in use among engineers are to be used with caution, as in many of them the factor of safety adopted does not allow a sufficient margin, while others are entirely incorrect. The figures given by Haswell, p. 474, are from 50 to 100 per cent. higher than those of Francis; while the results given by Haslett, p. 142, are evidently wrong, as a column 10 inches in diameter, 24 feet long, and an inch thick, has a strength of 73.800 pounds, while a pillar of the same section and 4 feet long, has a strength of only 78.600 pounds. Nystrom's table, Pocket Book of Mechanics, p. 269, gives the safe weight for a cast iron column 20 feet long, 12 inches outer diameter, and an inch thick, as 503 tons. Haswell gives the same as 150 tons, and Mr. Francis gives less than 100 tons. The breaking weight, by the most approved formula, is 618 tons. The tables of Mr. Trautwine, however, are thoroughly reliable, and being estimated for the breaking weight, allow of the application of any factor of safety desired.

post. For timber he allows 800 pounds per inch of the section for a post only six diameters in length, and 81 pounds only for a post of 60 diameters, the intermediate lengths being as shown in the following table:—

Length in Diameters.	Pounds.	Length in Diameters.	Pounds.	Length in Diameters.	Pounds.
6	800	24	368	42	166
8	760	26	328	44	151
10	720	28	296	46	138
12	680	30	269	48	127
14	640	32	246	50	117
16	600	34	227	52	108
18	560	36	210	54	100
20	479	38	195	57	90
22	416	40	183	60	81

The most convenient and reliable formula for the strength of columns is that prepared by Gordon, from the experiments of Hodgkinson, which, in its most general form, is as follows:—

$$P = \frac{f\,s}{1 + a\frac{l^2}{h^2}}$$

in which P is the breaking weight in pounds, s the sectional area, l the length, or height, and h the least external diameter, all in inches, and f and a constants obtained by experiment upon different materials.

From the above general formula Mr. Trautwine has prepared a valuable set of tables for columns of different forms, of cast and wrought iron, the formulæ for the several cases being as follows:— A representing the sectional area of the column, l the length, and d the least external diameter, all in inches, and w the breaking weight in pounds.

Hollow Wrought Iron Cylinder, $\dfrac{36.000\ A}{1 + \dfrac{l^2}{3000\ d^2}} = w.$

Solid Wrought Iron Cylinder, $\dfrac{36,000\ A}{1 + \dfrac{l^2}{2250\ d^2}} = w.$

Hollow Wrought Iron Square, $\dfrac{36\,000\ A}{1+\dfrac{l^2}{6000\ d^2}} = w.$

Solid Wrought Iron Square, $\dfrac{36.000\ A}{1+\dfrac{l^2}{3000\ d^2}} = w.$

Hollow Cast Iron Cylinder, $\dfrac{80.000\ A}{1+\dfrac{l^2}{400\ d^2}} = w.$

Solid Cast Iron Cylinder, $\dfrac{80.000\ A}{1+\dfrac{l^2}{200\ d^2}} = w.$

Hollow Cast Iron Square, $\dfrac{80.000\ A}{1+\dfrac{l^2}{533\ d^2}} = w.$

Solid Cast Iron Square, $\dfrac{80.000\ A}{1+\dfrac{l^2}{266\ d^2}} = w.$

The best adaptation of Gordon's formula to wooden posts is that made by Mr. Charles Shaler Smith, of Baltimore; it is as follows: —

$$\frac{5000\ A}{1+\left(\dfrac{l^2}{d^2}\times .004\right)}$$

in which d represents the side of a square post, or the least side of a rectangular one.

EXAMPLES.

Required the ultimate strength of a hollow cylindrical pillar of wrought iron 20 feet long, 10 inches external diameter, and one inch in thickness.

$$w = \frac{36{,}000\times\left[(10^2\times .7854)-(8^2\times .7854)\right]}{1+\dfrac{(20\times 12)^2}{3000\times 10\times 10}} = 853{,}792\ \text{lbs.}$$

Find the breaking weight of a rectangular wooden post 24 feet high, and 12 inches square.

$$w = \frac{5000\times 12\times 12}{1+\left(\dfrac{(24\times 12)^2}{12\times 12}\times .004\right)} = 217{,}918\ \text{lbs.}$$

Find the ultimate strength of a hollow cylindrical column of cast iron 18 feet high, 10 inches external diameter, and one inch thick.

$$w = \frac{80.000 \times \left[(10^2 \times .7854) - (8^2 \times .7854) \right]}{1 + \frac{(18 \times 12)^2}{400 \times 10 \times 10}} = 1,044,091 \text{ lbs.}$$

Experiments made upon the Phœnix columns show that the mode of flanging the parts together (page 273 and Plate XXIII.) increases so much the stiffness that the diameter may safely be taken from outside to outside of the flanges. The above formulæ apply to columns with the ends turned true and put up with care, and well bedded and fixed. For columns with round or hinged ends, or with ends rough from the foundry, so that the weight is not equally distributed, from 25 to 50 per cent. should be deducted from the amount given by the rules. It will be observed that the constant in the denominator for hollow cylinders is different from the constant for solid cylinders, being only half as great in the case of cast iron. This is evidently correct, but whether we can draw a precise line, on one side of which the cylinder is to be regarded as hollow, and on the other side solid, is doubtful. We may consider a cylinder a foot in diameter, with a hole in the centre of one inch diameter, as equivalent to a solid cylinder, as far as yielding by flexure is concerned. Mr. Trautwine takes the greatest thickness of metal in hollow columns as about one-eighth of the outer diameter. Mr. Hodgkinson gives one rule for what he terms "long columns," i. e., not less than 30 diameters for cast iron and wood, and not less than 60 diameters for wrought iron, and another rule for shorter ones. There can, of course, be no precise line drawn separating long from short columns. The real change from the column that yields by flexure alone to that yielding partly by flexure and partly by crushing, and from that again to one failing wholly by crushing, must be gradual, and the change in the formula should also be gradual.

THE STRENGTH OF HORIZONTAL BEAMS OR GIRDERS.

It has already been stated that the strength of a beam supported at the ends, and loaded in the middle, is directly as the breadth and as the square of the depth, and inversely as the length. The breaking weight of such a beam is expressed by the formula —

$$W = \frac{4\,b\,d^2\,S}{l},$$

in which b is the breadth, d the depth, and l the length, all in inches, and S an experimental value to be determined for different materials and for different forms of sections. For simple rectangular beams the value of S has been found as follows: —

White Pine,	1200
Oak,	1600
Cast Iron,	7500
Wrought Iron,	8250

For example, the ultimate strength of a beam of white pine 10 feet long, 4 inches wide, and 6 inches deep, will be —

$$\frac{4 \times 4 \times 6^2 \times 1200}{10 \times 12} = 5760 \text{ pounds.}$$

If the load, instead of being applied in the middle, is uniformly distributed, the strength is shown by the formula —

$$W = \frac{8\,b\,d^2\,S}{l};$$

or, in other words, a beam will bear twice as much when equally distributed over its length as it will when applied at the centre.

If the load is applied at one point, but not at the centre, the strength is found by the formula —

$$W = \frac{b\,d^2\,l\,S}{x\,y},$$

x and y being the two segments into which the length is divided

by the load. Thus, in the beam above, if the load is applied two feet from one end, the breaking load will be —

$$W = \frac{4 \times 6^2 \times 120 \times 1200}{24 \times 96} = 9000 \text{ pounds.}$$

As the strength of a girder depends upon the resistance of its fibres to extension and to compression, and as these fibres resist the more as they are farther removed from the neutral axis, we may very much increase the strength by removing the material from the centre to the top and the bottom. This is not practicable, of course, with *timber*, but it is with metal. If the resistance to compression and extension was the same in cast or wrought iron, then the top and bottom flanges of the girder should be equal; but as this is not the case in either of these metals, the beam will take an unsymmetrical form, the areas of the flanges being inversely as their unit of resistance. Thus a cast iron girder should have about six times the area in the lower flange that it does in the upper, while a girder of wrought iron should have about 50 per cent. more material in the top than in the bottom.

Mr. Hodgkinson's formula for a cast iron girder the lower flange of which contains six times as much metal as the top one is thus : —

$$26 \frac{a\,d}{l} = W\,;^*$$

in which W is the breaking weight in tons at the centre,

d the depth in inches,
a the area of the lower flange,
l the length in inches.

Thus a girder 20 feet long, 20 inches deep, and having an area of 40 square inches in the lower flange, will have a breaking load of —

$$26 \frac{40 \times 20}{240}, \text{ or } 86.66 \text{ tons.}$$

* The above coefficient (26) applies to girders cast with the bottom flange up. When the girder is cast upon its side, the coefficient is 24.

The direct strain upon the flanges is found by regarding each half of the beam as a bent lever, supporting one half of the load at its end, and by multiplying the weight by the half length, and dividing by the depth, or by multiplying the whole load at the centre by the length, and dividing by four times the depth. This may be expressed as below, the load being applied at the centre of the girder.

$$T = \frac{W \times l}{4\,d}.$$

If the load is uniformly distributed, the expression will be—

$$T = \frac{W \times l}{8\,d}.$$

In the example above this becomes $T = \frac{86.6 \times 240}{4 \times 20} = 260$ tons, or 6.5 tons of tension per inch upon the lower chord; and the area of the top flange being one sixth that of the lower one, the compressive strain upon that flange is 39.0 tons per inch.

In a wrought iron girder the top flange should be larger than the bottom one in the proportion of 60,000 to 36,000, or 40,000; or in the inverse ratio of the tensile and compressive resistances.

The formulæ above may be transformed so as to give the depth, or the flange area, the load being given: thus, in the first formula above, for rectangular beams, we have

$$W = \frac{4\,b\,d^2\,S}{l},$$

whence

$$b = \frac{W\,l}{4\,d^2\,S},$$

and

$$d = \sqrt{\frac{W\,l}{4\,b\,S}}.$$

So, too, if in the formula

$$T = \frac{W\,l}{4\,d},$$

we have given the load and the length, we may get the depth by transforming the above to

$$d = \frac{W\,l}{4\,T}.$$

In a girder with parallel flanges, the horizontal strain at any point is greatest when the load is over that point. Moreover, the strain is proportional to the rectangle of the segments into which the load divides the length of the beam, which rectangle is greatest when the segments are equal, i. e., when the load is in the middle. The strain upon the flanges of a girder is greatest at the centre, and decreases gradually towards the ends, where it disappears. We might, therefore, reduce the depth from the centre to the ends, thus making the strain upon the flanges uniform throughout.

Additional remarks upon the use of timber girders, and upon the form and dimensions of those of cast and wrought iron, with their application to railway work, will be given in advance.

Proof Strains.

It is often the custom to subject the metal to be employed in bridge work to a test, or proof strain. This proof may have for its object either the finding of the ultimate strength of the material, or its toughness and suitability for the particular purpose for which it is intended. Absolute strength is not the only qualification for the iron employed in bridge building. An iron may be exceedingly strong, but very hard and brittle, and altogether incapable of sustaining a shock; or it may be soft and ductile, and may thus draw out under a load so much as to unfit it for service. An iron for bridge work should neither be so hard as to be liable to fracture from a sudden blow, nor so soft as to stretch beyond a certain amount. If the ultimate strength of a specimen is to be found, we have only to place it carefully in the testing machine, and increase the strain slowly until we break it. If we wish, on the other hand, simply to test it for a special purpose, for which it is to be employed afterwards, we must not damage the material. The proof strain of the best wrought iron should not exceed the practical limit of elasticity, i. e., about 10 tons per square inch, or double the working load. Professor Rankine ob-

serves that we may approximate to the proof strength by applying and removing a moderate load for a number of times, observing carefully, meanwhile, whether the distortion of the piece increases sensibly, or not, from the repeated application of the load. By increasing the load, a weight will at length be reached, the successive applications of which will produce an increasing distortion, thus indicating that the proof strain has been passed. Mr. Stoney states that in his specifications he requires the set of tension plates (after fracture), when torn with the grain, to be not less than 5 per cent. of the original length; at right angles to the grain the set is generally much less — from 1 to 2 per cent. Too much stretching will make the iron harder and stronger, but brittle and less ductile. The wrought iron work for the Quincy Bridge was subjected to the following tests: From each lot of 100 merchantable bars five were indiscriminately selected, from each of which a piece not less than a foot long was cut; if either of these pieces failed under less than 50,000 pounds per inch, the whole lot was rejected. Afterwards, all of the bars were tested to 20,000 pounds per inch, and while under tension were struck from 2 to 4 blows, with hammers varying from 3 to 10 pounds, according to the size of the bar to be tested, and any bar which under this treatment showed a permanent set was rejected.

Original and Fractured Areas.

Mr. Kirkaldy lays great stress upon the importance of regarding the *fractured area* of the specimen, instead of the *original area*, before it was reduced by the elongation of the bar. "It seems most remarkable," he observes, "that an element of the highest importance should have been so long overlooked, namely, the contraction of the specimen's area when subjected to considerable strain, and the still greater contraction, at the point of rupture, which takes place in a greater or less degree as the material is soft or hard, and the consequent influence this reduction must have on the amount of weight sustained by the speci-

men before breaking. The apparent mystery of a very inferior description of iron suspending, under a steady load, fully a third more than a very superior kind, vanishes at once when we find that the former had the benefit of retaining to the last its original area, only slightly decreased, whilst the latter on breaking was reduced very nearly to a fourth of its original area — the one a hard and brittle iron, liable to snap suddenly under a jerk or blow, the other very soft and tough, impossible to break otherwise than by tearing slowly asunder." * "The working strain," Mr. Kirkaldy remarks, "should be in proportion to the breaking strain per square inch of *fractured area*, and not to the breaking strain per square inch of *original area*, as heretofore." The softness has the effect of lessening the amount of the *breaking* strain, but it has the opposite effect as regards the *working* strain, since the soft irons are less liable to snap than the hard ones, and thus are much more suitable for structures subjected to vibrations and concussions, like railway bridges.†

Expansion and Contraction of Metals.

In applying iron or steel to bridge work, regard must be had to the effect of heat and cold upon the metal. Cast iron expands .0000062 of its length for each degree Fahr., wrought iron .0000069, and steel, .0000064.‡ The effect of a tensile or compressive strain is also to be considered. Cast iron is compressed .0000000804 of its length, for each pound of compressive force, and is extended .000000107 of its length for each pound of tension; so, too, wrought iron is compressed .0000000446, and is extended .0000000357 per pound.

* Experiments on Wrought Iron and Steel, pp. 24 and 96, 97.

† These remarks apply to extension, but not to compression.

‡ Mr. Chanute, Kansas City Bridge, p. 109, gives the coefficient of expansion of pine wood for one degree Fahr. as .00000227.

Concluding Rules for Practice.

From the preceding figures the following general conclusions are drawn, showing the principal characteristics of the materials used in bridge building.

Timber.

The ultimate tensile strength of oak is from 10,000 to 14,000 pounds per square inch; the compressive strength, from 4200 to 7700 pounds; the lateral adhesion, 2300 pounds; and the shearing strength in the direction of the fibre, about the same. The shearing strength across the grain, as in treenails, is 4000 pounds per inch. The value of S, in the formula for transverse strain, is 1600; the value of f, in the formula for columns, 7200; and that of a, $\frac{1}{250}$, or .004. The weight of a cubic foot is from 50 pounds when dry to 75 pounds when green. The ultimate tensile strength of pine is from 10,000 to 12,000 pounds per inch; the compressive strength, from 5300 to 6700 pounds; the lateral adhesion, from 500 to 1000 pounds; and the shearing strength, in the direction of the fibre, about the same. The value of S is 1200, and that of f and a the same as for oak.* The weight of a cubic foot varies from 30 pounds when dry to 50 pounds when green. The factor of safety for wood ranges from 6 to 12. For tension we may use from 800 to 1000 pounds; for compression, from 600 to 800 pounds; for shearing along the grain, from 100 to 125 for pine, and 400 for oak. For crushing across the fibres 300 pounds for pine, and 500 for oak, may be employed; and for the shearing of white oak treenails, 600 pounds per inch.

Cast Iron.

Ultimate tensile strength from 15,000 pounds, for common iron,

* Hodgkinson's formulæ for timber pillars, as given by Mr. Stoney, Vol. II. p. 196, give one constant for oak, and another for pine. Mr. Francis also adopts the two constants for the different kinds of wood. (Cast Iron Pillars, p. 25.) Hodgkinson's rule, as given by Professor Rankine (Civ. Eng., p. 238), employs one constant for both; and Mr. Gordon's formula gives only one constant for timber.

to 25,000 for toughened. Compressive strength from 80,000 pounds for common iron to 144,000 for toughened. Value of S, for transverse strain, is 7500; the value of f in the formula for columns, 80,000; and that of a $\frac{1}{400}$, or .0025. Weight per cubic foot, 450 pounds. Expansion per degree Fahr. .0000062. Extension per pound of tensile force, .0000000804. Compression per pound, 000000107. Factor of safety, from 5 to 10.

Wrought Iron.

Tensile strength, rolled bars, 57,000 to 68,000 pounds. Angle iron, etc., 54,000 to 63,000 pounds. Plates, lengthwise, 50,000 to 62,000 pounds. Plates, crosswise, 46,000 to 60,000 pounds. Compressive strength from 30,000 to 40,000 pounds. Shearing strength about the same as the tensile strength. Expansion per degree Fahr., .0000069. Extension per pound, .0000000357. Compression per pound, .0000000446. Weight per cubic foot, 480 pounds. Value of S, for transverse strain, 8250. f, in formula for posts, 36.000; and a $\frac{1}{3000}$ or .00033. The tensile strength of the best iron wire ranges from 80,000 to 100,000 pounds per square inch. Factor of safety, from 5 to 8.

Steel.

Tensile strength of steel bars, 60,000 to 132,000 pounds. Plates, lengthwise, 75,000 to 102,000 pounds. Plates, crosswise, 67,000 to 97,000 pounds. Crushing strength from 200,000 to 400,000 pounds. Shearing strength, $\frac{3}{4}$ of the tensile strength. Expansion per degree of Fahr. .0000064. Weight per cubic foot, 490 pounds. So little use has yet been made of steel in bridge work that its factor of safety is not very definitely determined. Much of the steel made by the new processes has been too hard and too uneven in its strength to be relied on. The great resistance, however, both to extension and compression, which this metal possesses, makes it especially applicable to work where lightness is essential. There can be no doubt that whenever a uniform, reliable, tough and strong steel is required for bridge building, the

demand will be supplied. Of such steel the proportion of the ultimate strength that may be safely employed in practice will certainly not be less than that of wrought iron; and it will most likely be more. "Probably," says Mr. Stoney (Vol. II. p. 248), "one fourth of the tearing strain, or from 8 to 10 tons per square inch for plates, is a safe tensile working strain." Again (upon p. 39 of the same work) it is remarked that, "The crushing strength of steel is so high that 12 or 15 tons per square inch is perhaps a safe compressive working strain, when the material is not allowed to deflect."

CHAPTER IX.

JOINTS FOR CONNECTING PARTS OF BRIDGES.

It is laid down as a maxim in most works upon construction, that a joint should be so made as to be as strong as the parts it serves to connect. This, however, cannot always be done. The best splices for timber must necessarily weaken the material. The same is true in the best form of riveted joint, unless the bars or plates to be connected are thickened at the ends or edges — an operation seldom worth performing. It should, however, be borne in mind, that the strength of any structure is limited by the strength of its weakest part; and especial care should therefore be taken in arranging splices and joints to reduce the strength as little as possible. Attention should also be paid to the varying amount of strain in different parts of any structure, the strength of the joints being varied according to the strain put on them. Tension joints at the centre of the chords of a bridge need to be stronger than those near the ends. Joints should, if possible, be so contrived as not to be subjected to different kinds of strain, as the best joint for one strain is rarely the best for another.

Joints in Timber Work.

There is little to be said in regard to compression joints in timber. The pieces should be brought together with a carefully made plain butt joint, and fastened to prevent lateral movement. In the case of the top chords of bridges the packing blocks may be cut into the stringers by a small amount, to insure the various

timbers acting together as one piece. The indentation for this purpose need not be more than an inch in depth; and when the timber is sawed very straight and true, less will answer. The length of the block at the joint may be from 4 to 6 times the depth, and intermediate blocks may be square.

It is the tension joint in timber work that requires to be carefully proportioned, in order to preserve as far as possible the full strength. Perhaps the simplest splice would be made as in Fig. 42, where the timbers are brought together with a butt

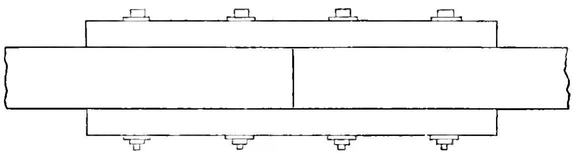


Fig. 42.

joint, and fished. In this case the joint strength of the fish bars, the tensile strength of the timber itself, the collective shearing strength of the bolts on each side of the joint, and the resistance to dragging the bolts endwise through the wood, should be equal. Such a joint, however, would seldom be employed. A more common and a better one is shown in Fig. 43, the splices

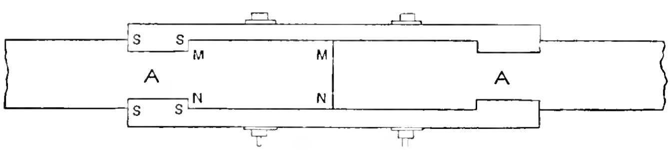


Fig. 43.

being of hard wood. In this joint the tensile strength of the timber at the weakest point, A, the shearing resistance at M M and N N, and finally the shearing strength of the blocks at S S and S S, should be equal. If the resistance to crushing were reckoned equal to that of tension, then the areas of the shoulders at M and N should together be equal to the sectional area at A; or, in other words, we should cut the sticks half off, reducing the ten-

sile strength 50 per cent. This, of course, would not do. We may, however, get the required shoulder area in another way, with a much less reduction of the tensile strength, viz., by multiplying the number of shoulders as in Fig. 44. The number of

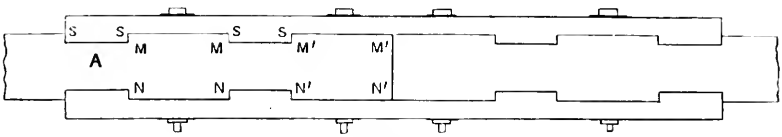


Fig. 44.

shoulders being doubled, the depth need be only half as much. With regard to the length M M, or N N, calling the shearing resistance of pine 100 pounds, and the tensile strength 1000 pounds, the shearing area must be ten times as much as the tensile area, or ten times as much as the crushing or shoulder area; and as the depth of the beam is a common factor in all of the several areas, the length M M, Fig. 43, must be simply ten times the depth of the shoulder; or M M, plus N N, must be ten times the reduced sectional breadth at A. In Fig. 44, M M plus M′ M′ added to N N plus N′ N′, must be ten times the reduced breadth at A. This splice may be arranged otherwise, as in Fig. 45, in which

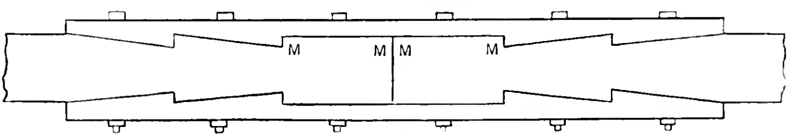


Fig. 45.

the distances M M are ten times the depth of shoulder. The bolts should be put through the splice at the points shown in the Figs., and not through the most reduced section. The joint strength of the two splices at the centre should be equal to that of the timber at its weakest point.

The joint shown in Fig. 46 has been employed for the lower chords of bridges. Here the rods should have a strength equal to the net strength of the timber where it is cut to admit the

block. The double shear from the block to the joint should also have the same resistance. The block should be narrow, so as not to reduce too much the timber, but should be deep, in the

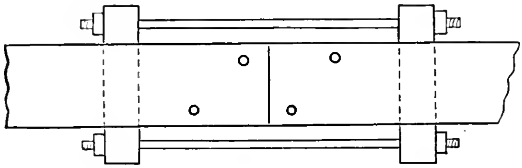


Fig. 46.

length of the stick, to have the requisite transverse strength. Another joint, proposed by Mr. Whipple (Bridge Building, p. 100, Fig. 32, Plate VI.), is shown in Fig. 47. This joint, however,

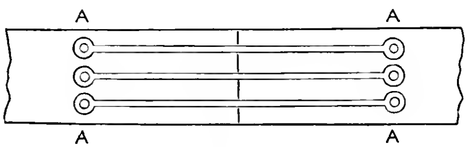


Fig. 47.

may be improved. In Fig. 47 the chord is reduced by three bolt holes. In Fig. 48 the section at any one point is less reduced, supposing the bolts to be in both cases all of a size. If, how-

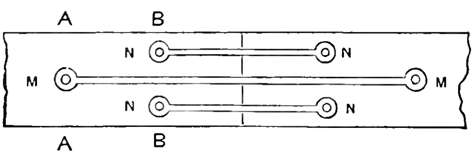


Fig. 48.

ever, we make the bolts N N N N, in Fig. 48, smaller than those in Fig. 47, and those at M M larger, we may keep the same gross area of bolt section, and cut the timber less. The section on the line A A will be reduced by simply the one bolt hole: at B B the timber is reduced by two smaller holes; but before the stick can break at that place the rod M M must break, or else the bolt

M must shear, or pull out by shearing the timber. The several bolts should be at a distance from the joint proportioned to their size, the smaller ones being nearer than the large one. The section of the several rods, or bars, should also be proportioned to the shearing strength of the bolts at their ends.

Joints for Iron Work.

The two principal modes of connection employed for structures of wrought iron are the riveted and the pin joint. The riveted joint is less used now than formerly, as the older structures of boiler plate are giving place to the lighter and more economical open work girders, or trussed frames.

A riveted joint equally strong with the entire plate cannot be made, but a near approximation to the full strength may be obtained by a judicious arrangement of rivets. The strength of this kind of connection depends upon the tensile strength of the plate, and upon the shearing resistance of the rivets. The following statements, by Mr. Latham, exhibit briefly the principles of riveted joints.*

The efficiency of a plate is the strain that may safely be put upon it.

The line of fracture is a line crossing the strain in such a way that the section of the plate along it is a minimum.

The breaking area is the section along the line of fracture.

The efficiency of a plate in tension will be taken at $4\frac{1}{2}$ tons per square inch of the breaking area.

The efficiency of a plate in compression would be 4 tons per inch of the full section if the rivet holes were filled completely. In practice this is not the case, yet the plate is not weakened to the full extent of the holes. The efficiency will thus be taken at $4\frac{1}{2}$ tons per inch along the line of fracture.

* The Construction of Wrought Iron Bridges. By John Herbert Latham. Cambridge: Macmillan. 1858. An exceedingly valuable work, but not easily obtained.

The effective *line of bearing* of a rivet is equal to the diameter of the rivet.

The effective bearing area of a rivet is equal to the diameter into the thickness of the plate.

The efficiency of the bearing surface of wrought iron will be taken at 5 tons per square inch.

Thus the line of bearing represents the same degree of efficiency, in the same plate, as a line of fracture $\frac{10}{9}$ of its length, since it will bear 5 tons where the latter will bear $4\frac{1}{2}$.

The efficiency of a rivet is proportional to its area, whether it resists shearing or tension; and the resistance to these two strains by wrought iron is practically the same, and may be taken at from 4 to 5 tons per inch.

In a good joint the friction of two plates riveted together is so great that for a time it takes the whole strain, so that the plates would not slide even if the rivets did not fill the holes. This applies, however, only to fresh joints. About 15° Fahrenheit causes the same extension of wrought iron as one ton of tension per inch. If then a rivet cools 360°, after riveting, it would induce a tension of $\frac{360}{15}$ or 24 tons per square inch of its section. The coefficient of friction of wrought iron on wrought iron is about $4\frac{1}{3}$, so that if the rivets at a joint are at a tension of 24 tons per inch, the friction is $\frac{24}{4\frac{1}{3}}$ or about $5\frac{1}{2}$ tons per square inch of section of the rivets. If a joint remained undisturbed this friction might remain, even until the plates were amalgamated; but in the case of bridges, subjected as they are to vibration and to changes of temperature, this friction cannot be counted on at all.*

* Mr. Latham is doubtless quite right in neglecting entirely the value of friction as of any practical use. Mr. Edwin Clark, in his work upon the Britannia and Conway Bridges, refers to certain experiments in which the rivet holes were purposely cut out much larger than the rivet, and the friction was experimentally shown to be sufficient to sustain the working load without the aid of the shearing resistance of the rivets; and he remarks that "considerable strength is obtained from the friction produced by the cooling of the rivet, and hence it is not requisite that the area of the rivet should equal that

Illustrations of Riveted Joints.

The preceding remarks are illustrated by the following examples given by Mr. Latham. Let the plate in Fig. 49 be 8 inches

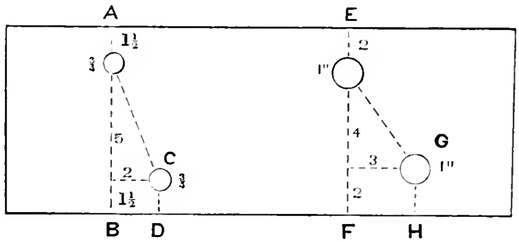


Fig. 49.

wide and half an inch thick. The efficiency of the whole plate will be—

$$8 \times \frac{1}{2} \times 4\frac{1}{2} = 18 \text{ tons.}$$

Along the line A B the value of the section will be—

$$8 - \frac{3}{4} = 7\frac{1}{4}, \text{ and } 7\frac{1}{4} \times \frac{1}{2} \times 4\frac{1}{2} = 16.3 \text{ tons.}$$

Again along the line A C D we shall have the value—

$$3 + \sqrt{25 + 4} - 1\frac{1}{2} = 6\frac{7}{8}, \text{ and } 6\frac{7}{8} \times \frac{1}{2} \times 4\frac{1}{2} = 15.5 \text{ tons.}$$

In the same manner the efficiency on the line E F is—

of the plates united, as with a simple loose pin:" and, again; that "by judicious riveting the friction may, in many cases, be nearly sufficient to counterbalance the weakening of the plate by punching."

Mr. Humber, in his large work, places no reliance upon friction, since, he observes, it is very liable to become reduced, and even entirely destroyed, by vibration. Mr. Stoney remarks that it does not follow that the ultimate strength of the joint is increased by the friction, though the latter may be sufficient to convey the working strain without subjecting the rivets to shearing. (Vol. II. p. 361.) Experiments made upon newly riveted plates cause the rivets to appear stronger than they really are, since the friction of the plates and the shearing resistance of the rivets act together.

$$8 - 1 = 7, \text{ and } 7 \times \tfrac{1}{2} \times 4\tfrac{1}{2} = 15\tfrac{3}{4} \text{ tons.}$$

And finally upon the line E G H we have the value —

$$4 + \sqrt{16 + 9} - 2 = 7, \text{ and } 7 \times \tfrac{1}{2} \times 4\tfrac{1}{2} = 15\tfrac{3}{4} \text{ tons.}$$

Thus, by removing the rivet G to a sufficient distance from the line E F, we may render the efficiency of the plate along E G H the same as that along E F, while the shearing efficiency of the rivets is doubled.

Let us have again the arrangement given on the line A A′, Fig. 50. Calling the plate 9 inches wide, and a fourth of an inch

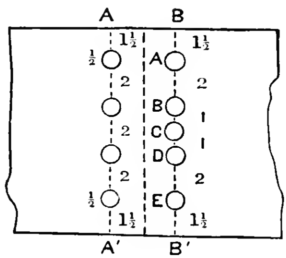


Fig. 50.

thick, and the rivets half an inch, the efficiency of the plate will be —

$$9 - (4 \times \tfrac{1}{2}) = 7, \text{ and } 7 \times \tfrac{1}{4} \times 4\tfrac{1}{2} = 7.875 \text{ tons.}$$

The bearing efficiency of the rivets in the same line is —

$$4 \times \tfrac{1}{2} \times \tfrac{1}{4} \times 5 \text{ T} = 2.5 \text{ tons.}$$

And the shearing efficiency of the rivets on the same line is —

$$4 \times \tfrac{1}{2} \times \tfrac{1}{2} \times .7854 \times 4\tfrac{1}{2} = 3.53 \text{ tons.}$$

Thus the bearing efficiency of the rivets limits the strength of the joint, and the resistance to shearing need not be considered. The importance of properly regarding the bearing surface of the rivets is strikingly shown in some experiments by Mr. Fairbairn, from which it appears that the tensile resistance of the plate

varied from 19 to 27 tons per square inch, according as the bearing surface of the rivets was less or greater. This point will be referred to in advance in considering the action of a pin upon the eye of a tension bar.

If, in the arrangement shown in Fig. 50, upon the line B B′ we put an extra rivet in at C, the joint will yield by the failure of the *plate* between B and D, and from a lack of bearing area at the rivets A and E; and we should have—

$$\text{Rivets } A + E + \tfrac{1}{2}(B + D) = \tfrac{3}{2} \times \tfrac{1}{4} \times 5 \text{ tons} = 1\tfrac{7}{8} \text{ tons.}$$
$$\text{Rivets } C + \tfrac{1}{2}(B + D) = 1 \times \tfrac{1}{4} \times 5 \text{ tons} = 1\tfrac{1}{4} \text{ tons.}$$
$$\text{Plate from } B \text{ to } D = 1 \times \tfrac{1}{4} \times 4\tfrac{1}{2} \text{ tons} = 1\tfrac{1}{8} \text{ tons.}$$

Thus between B and D the value of the *plate* limits the efficiency of the joint; and the whole strength will be—

$$1\tfrac{7}{8} + 1\tfrac{1}{8} = 3 \text{ tons.}$$

If the plate in Fig. 51, is 9 inches wide, and breaks through

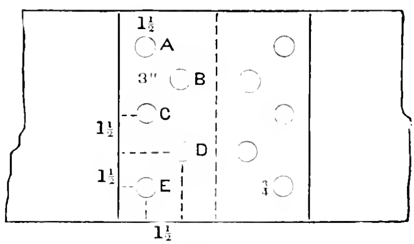


Fig. 51.

A C E, the line of fracture is—

$$9 - (3 \times \tfrac{3}{4}) = 6\tfrac{3}{4} \text{ inches.}$$

If the plate or cover breaks through A B C D E, the line of fracture is—

$$1\tfrac{1}{2} + 1\tfrac{1}{2}\sqrt{2} \times 4 + 1\tfrac{1}{2} - 5 \times \tfrac{3}{4} = 7\tfrac{3}{4} \text{ inches.}$$

If the cover breaks through A B D E, the line of fracture is—

$$1\tfrac{1}{2} + 1\tfrac{1}{2}\sqrt{2} + 3 + 1\tfrac{1}{2}\sqrt{2} + 1\tfrac{1}{2} - 4 \times \tfrac{3}{4} = 7\tfrac{1}{4} \text{ inches.}$$

If the rivets all cut the plate, the line of bearing is —

$$\tfrac{3}{4} \times 5 = 3\tfrac{3}{4} \text{ inches.}$$

The latter being much less than either of the former values shows the joint to be defective in rivets. It may be improved as in

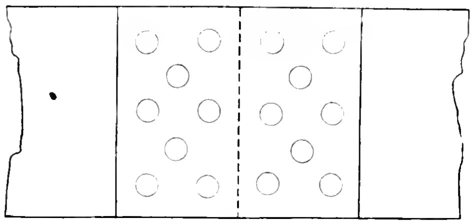


Fig. 52.

Fig. 52, in which the value of the line of bearing of the rivets is —

$$8 \times \tfrac{3}{4} = 6,$$

which, multiplied by the bearing efficiency, 5 tons, gives very nearly the same results as the $6\frac{3}{4}$ inches above (through A C E) multiplied by the tearing efficiency, $4\frac{1}{2}$ tons, or about three fourths of the full value of the plate.

In Fig. 53 the $\frac{7}{8}$ inch rivets have 2 inches between centres, and

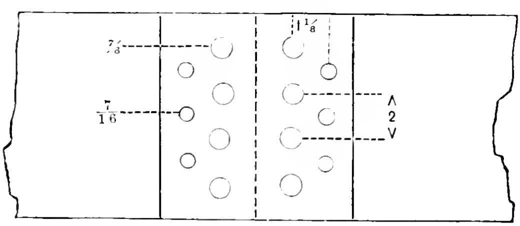


Fig. 53.

thus $1\frac{1}{8}$ inches between rivets, and the equivalent line of bearing being $\frac{9}{10}$ as much, is $\frac{9}{10}$ of $1\frac{1}{8}$, or 1 inch, very nearly ; so that the strength of the plate is more than the bearing strength of the

rivets. Add now the $\frac{7}{16}$ inch rivets, as in the Fig., and the plate section between two $\frac{7}{16}$ rivets and a $\frac{7}{8}$ rivet is—

$$2\sqrt{\tfrac{9}{8}]^2 + 1^2} - \tfrac{7}{8} - \tfrac{7}{16} = 1\tfrac{11}{16} \text{ inches,}$$

which is equivalent to a line of bearing of—

$$1\tfrac{11}{16} \times \tfrac{9}{10}, \text{ or } \tfrac{27}{16} \times \tfrac{9}{10}, \text{ or } 1.52 \text{ inches ;}$$

while the amount of bearing acting upon this section is due to two rivets, viz.,—

$$\tfrac{7}{8} + \tfrac{1}{2} \text{ of } \tfrac{7}{16} + \tfrac{1}{2} \text{ of } \tfrac{7}{16} \text{ or } \tfrac{7}{8} + \tfrac{7}{16} = 1\tfrac{5}{16} \text{ inches ;}$$

which, compared with the $1\frac{11}{16}$ inches above, shows that the plate is still able to cut all of the rivets. Thus the efficiency of the joint, as far as the plate is concerned, is determined by its hold on the rivets, and is, per foot run of the joint, the plate being $\frac{1}{4}$ inch, and the cover $\frac{3}{8}$ in thickness—

$$\tfrac{7}{8} + \tfrac{7}{16} \text{ or } \tfrac{21}{16} \times \tfrac{1}{4} \times 6 \text{ (rivets)} \times 5 \text{ (tons)} = 9.84 \text{ tons.}$$

The value of the cover between the $\frac{7}{8}$ inch rivets is per foot run—

$$12 - 6 \text{ (rivets)} \times \tfrac{7}{8} \text{ (diam.)} \times \tfrac{3}{8} \times 4\tfrac{1}{2} \text{ (tons)} = 11.4 \text{ tons.}$$

Hence the value of the joint, per foot run, is determined by the hold of the *plate* upon the rivets, which is 9.84 tons; and the whole plate being $12 \times \frac{1}{4} \times 4\frac{1}{2}$ or $13\frac{1}{2}$ tons, the joint has $\frac{984}{1350}$ or 73 per cent. of the whole strength of the plate.

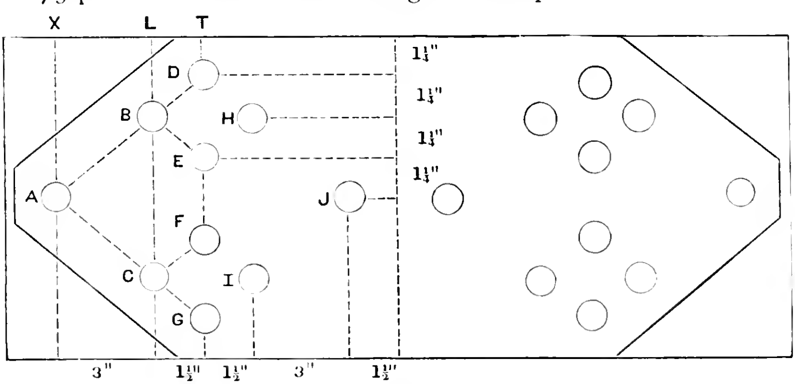


Fig. 54.

Suppose we have the joint shown in Fig. 54; the plate and the cover each being half an inch thick, and the rivets $\frac{3}{4} + \frac{1}{16}$ or $\frac{13}{16}$ in diameter. The line of fracture through A will be —

$$10 - \tfrac{13}{16}, \text{ or } 9\tfrac{3}{16}. \qquad \text{(A.)}$$

So long as the disposition of rivets is such that A B + B L is greater than A X, and again when B D + D T is greater than B L, the weak line of the joint will still be that through A. If the joint fails by tearing A, and going through B and C, we shall have for the actual plate section —

$$10 - (2 \times \tfrac{13}{16}) = 8\tfrac{3}{8}.$$

The plate section equivalent to the bearing of the rivet A is —

$$\tfrac{3}{4} + \tfrac{1}{16} \times \tfrac{10}{9} = \tfrac{7}{8} + \tfrac{1}{32};$$

and the sum of the two is —

$$9 + \tfrac{1}{4} + \tfrac{1}{32}. \qquad \text{(B.)}$$

If the joint fails by tearing A, and going through D B E F C G, we shall have for the plate —

$$5 + 4\sqrt{\tfrac{5}{4})^2 + \tfrac{3}{2})^2} - 6 \times \tfrac{13}{16} = 7\tfrac{7}{8} + \tfrac{1}{16};$$

and adding to this the value of the rivet A, as above, we have —

$$7\tfrac{7}{8} + \tfrac{1}{16} + \tfrac{7}{8} + \tfrac{1}{32} = 8\tfrac{3}{4} + \tfrac{1}{16} + \tfrac{1}{32}. \qquad \text{(C.)}$$

If the joint fails by tearing A, and going through L B E F, etc., we shall have —

$$5 + 2\tfrac{1}{2} + 2\sqrt{\tfrac{3}{2})^2 + \tfrac{5}{4})^2} - 4 \times \tfrac{13}{16} = 8\tfrac{1}{8}:$$

adding to this the value of the rivet A, as above, we have —

$$8\tfrac{1}{8} + \tfrac{7}{8} + \tfrac{1}{32} = 9\tfrac{1}{32}. \qquad \text{(D.)}$$

If the plate were to be cut by the rivets A B C, and to break through D E F G, we should have for the plate —

$$10 - (4 \times \tfrac{13}{16}) = 6\tfrac{3}{4}\,;$$

and the length equivalent to the bearing of the rivets A B C —

$$3 \times \tfrac{13}{16} \times \tfrac{10}{9} = 2\tfrac{3}{4}.$$

The sum is

$$6\tfrac{3}{4} + 2\tfrac{3}{4}, \text{ or } 9\tfrac{1}{2}. \qquad \text{(E.)}$$

If all the rivets cut their bearings, the equivalent line of section will be —

$$10 \times \tfrac{13}{16} \times \tfrac{10}{9}, \text{ or } 9.02. \qquad \text{(F.)}$$

Thus the strength of the joint is (C above) $8\frac{27}{32}$, or 8.84, or $\frac{884}{1000}$ of the full plate.*

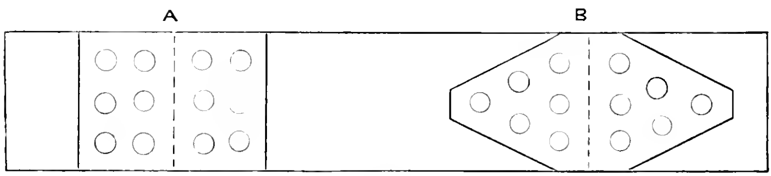


Fig. 55.

In Fig. 55 a joint in common use is shown, at A, by which an unnecessary amount of plate is punched out by the outer line of rivets. The joint at B reduces the section of the plate by only one third as much, while giving an equal number of rivets. In Fig. 56 are shown two joints; one at A, and another at B; the

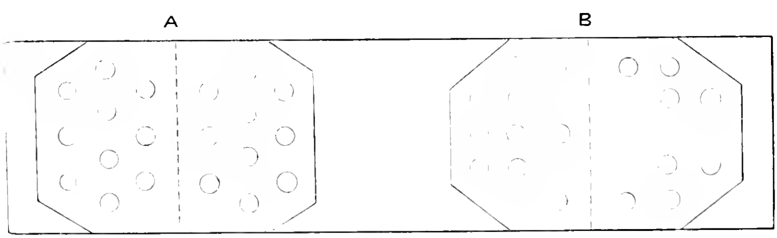


Fig. 56.

* In practice, a full sized sketch of any joint proposed will be found convenient for obtaining the length of the lines referred to in the above examples.

latter showing two different forms, in all of which the outer and the inner rows of rivets are reduced in number. In these joints the strength of the plate is limited by the section through the outer row of rivets, the strength of the cover by the section through the inner row. In Fig. 57, again, the connections are so

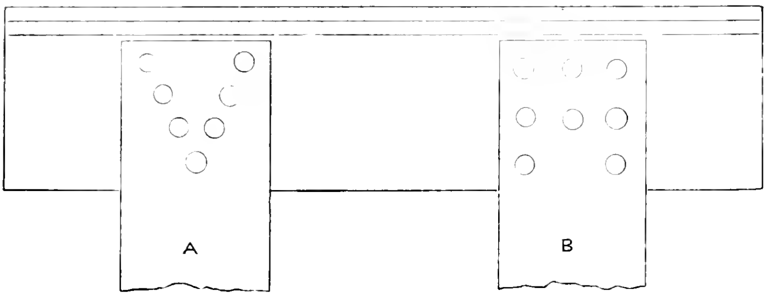


Fig. 57.

made as to reduce the strength of A by one rivet only, and that of B by two. In Fig. 58 the tension member at B is reduced by

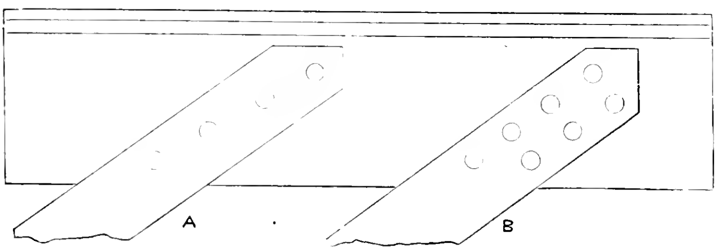


Fig. 58.

one rivet only, and is thus equally strong with the attachment at A, while having more rivet resistance. Additional illustrations will be found in the plate representing the Canestota Lattice Bridge, upon the New York Central Railroad, by Mr. Hilton.

Chain Riveting.

It was imagined by Mr. Fairbairn that the disposition of rivets shown at A, Fig. 55, was much superior to that at B; that is, that

rivets should lie in the same line of tension, thus forming what he has denominated "Chain Riveting." It is, however, plain to see from what has preceded that such is not the case.*

In Fig. 59 several forms of riveted joint are shown. A is the single lap, in which the rivets are subjected to shearing in one

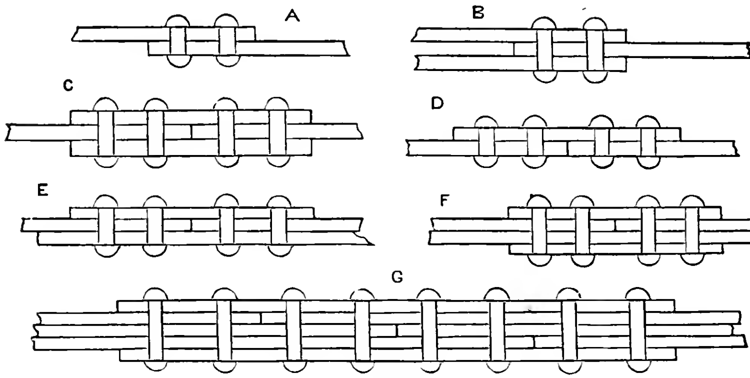


Fig. 59.

place only. B is a double lap, in which the rivets are subjected to a double shear. As far, therefore, as shearing alone is concerned, B would require only half the number of rivets that would be needed in A. The bearing efficiency, however, is not increased in B. D shows a butt joint, with a single welt, or cover; here, too, the rivets are in single shear. C shows a double welt butt joint, the rivets being in double shear. The action of the strains is always more direct in the double than in the single joints. E and F represent two joints, the first being that

* "This assumption" (that chain riveting was superior), says Mr. Edwin Clark, "arose from considering a plate as a number of parallel fibres independent of each other, and thus when one line of fibres was once destroyed by a rivet hole, it was conceived that other rivets in the same line would not further weaken the plate; whereas with zigzag riveting another intermediate line of fibres is destroyed, so that a plate thus riveted would be very weak, all its lines of fibres being separated. This conclusion, it will be shown by future experiments, was proved to be entirely erroneous; the zigzag riveting being the stronger."—*Britannia and Conway Tubular Bridges.* Vol. II. p. 519.

employed for the tension riveting in the first of the Conway tubes ; the second is that adopted in the second Conway tube, and in the Britannia Bridge.

Mr. Clark observes, with regard to these joints, that in the first the tendency of the jointed plate to separate is resisted only by the rivets, which bear partly on the cover and partly on the adjacent plate, and that whatever be the amount of strain thus imposed on the whole plate, it is additional to the strain naturally sustained by that plate from its own position, and that this increased strain occurs just where the plate is weakened by the rivet holes, which are necessary for the attachment of the cover. To remedy this defect, the extra cover, shown in F, of half the thickness, was added.*

When several layers of plates are to be connected, the amount of cover plates required will be economized by bringing the joints near together, and making one pair of covers answer for all, as in Fig. 59, G.

If, in a compressed joint, the ends were planed truly, and brought into close contact, the only riveting required would be simply that needed to keep the plates in place ; but the ends are rarely made true enough for this, and the very process of riveting often draws the plates apart. As work is commonly done, the compression joint should be made strong enough to carry the thrust, and should have the same shearing and bearing areas as if in tension.

The amount punched out of the plate, however, does not affect a compression joint, the effective area of the plate being that which bears on the rivets. In many bridges the tension and compression joints have been made alike.

It is easy to determine, theoretically, the ratios between the thickness of plate and the diameter and distances of the rivets ; but such theoretical determinations are in most cases overruled by practical considerations. It is stated in many works that the

* Britannia and Conway Bridges. Vol. II. p. 562.

sectional area of the rivets in any joint should be equal to the net section of the plate.* As a general thing, a joint thus made would be deficient in bearing area. Only when,

$$d \times t \times 5 \text{ tons} = d^2 \times .7854 \times 4.5 \text{ tons}$$

will the shearing and bearing values be alike; and this will only occur when the thickness of the plate is 0.7 of the diameter of the rivet. Again, the theoretical distance from the edge of the plate at the joint back to the first line of rivets, in order to resist the shearing out of the rivet through the plate, is altogether too small. Finally, no formulæ take into account the weakening of the plate by punching.

Dimensions of Riveted Joints in Boiler Plate Work.

The proportions for riveted joints given by Mr. Fairbairn and others refer to boiler work, and not to girder work. A steam tight joint requires the rivets to be close to each other, and close to the edge of the plate. A girder joint does not. In the following table the thickness of the plate, the diameter of rivets, and the pitch for boiler work, are those deduced from a wide range of experience by Mr. Fairbairn. The pitch for girder work is obtained by multiplying that for boiler work by the numbers in col. 4. The distance from the joint to the first row of rivets is, in boiler work, generally equal to the diameter of the rivet; in girder work once and a half the same: —

* "The joint sectional area of the rivets should be equal to the sectional area of the plates left after punching the rivet holes." (Rankine, Civ. Eng., p. 515.) "The sectional area of all the rivets in a joint taken together should be equal to the effective section of the plate." (Humber, Strains in Girders, p. 58.) The above rules are badly expressed. Mr. Stoney (Vol. II. p. 354) gives a plainer rule: "When a joint connects tension plates, the aggregate shearing area of the rivets *on each side of the joint line* multiplied by the safe shearing unit strain of the rivets should equal the total working strain transmitted through the plates."

Thickness of Plate.	Diameter of Rivet.	Boiler Pitch.	Multiplier.	Girder Pitch.	Boiler Distance	Girder Distance.
$\frac{3}{16}$	$\frac{3}{8}$	$1\frac{1}{4}$	1	$1\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$
$\frac{1}{4}$	$\frac{1}{2}$	$1\frac{1}{2}$	1	$1\frac{1}{2}$	$\frac{1}{2}$	$\frac{3}{4}$
$\frac{5}{16}$	$\frac{5}{8}$	$1\frac{5}{8}$	$1\frac{1}{4}$	2	$\frac{5}{8}$	$\frac{7}{8}$
$\frac{3}{8}$	$\frac{3}{4}$	$1\frac{3}{4}$	$1\frac{1}{4}$	$2\frac{3}{16}$	$\frac{3}{4}$	$1\frac{1}{8}$
$\frac{1}{2}$	$\frac{13}{16}$	2	$1\frac{1}{4}$	$2\frac{1}{2}$	$\frac{3}{4}$	$1\frac{1}{8}$
$\frac{9}{16}$	$\frac{7}{8}$	$2\frac{1}{4}$	$1\frac{1}{2}$	$3\frac{3}{8}$	$\frac{7}{8}$	$1\frac{1}{4}$
$\frac{5}{8}$	$\frac{15}{16}$	$2\frac{1}{2}$	$1\frac{1}{2}$	$3\frac{3}{4}$	$\frac{7}{8}$	$1\frac{1}{4}$
$\frac{11}{16}$	1	$2\frac{3}{4}$	$1\frac{1}{2}$	$4\frac{1}{8}$	1	$1\frac{1}{2}$
$\frac{3}{4}$	$1\frac{1}{8}$	3	$1\frac{1}{2}$	$4\frac{1}{2}$	$1\frac{1}{8}$	$1\frac{3}{4}$

Thus the diameter of a rivet for plates less than half an inch in thickness is equal to twice the thickness; for plates more than half an inch, once and a half of the thickness. The length of a rivet before hammering down should be equal to the total thickness of the plates to be connected, added to twice and a half the rivet diameter. The diameter of a rivet head should be about twice that of the shank, and the height of the head not less

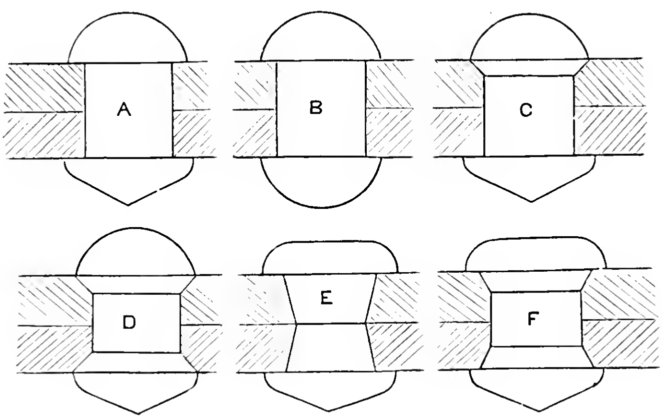


Fig. 60.

than half the diameter. Fig. 60 shows several forms for rivets A and B being those most commonly used for girder work. A

is the common hammer headed rivet; B, the snap headed rivet, the semi-spherical head being formed with a swage; C, hammer headed rivet, with one plate rimmed out; D, the same with both plates rimmed; E flat headed rivet with double conical shank, as used by Mr. Scott Russell; F, flat headed rivet with both plates rimmed. A rimmed hole allows a sounder rivet head, as the shank does not join the head at a right angle.

Effect of Punching Iron Plates.

All punching damages to some extent the plates, especially in hard varieties of iron and in steel. Mr. Fairbairn found iron plates to be reduced in strength about 20 per cent. by punching, exclusive of the weakening by diminishing the actual section. The punch should be rather larger than the thickness of the plate, in order not to damage the latter. The reduction of the strength of a plate by *drilling* the holes is very slight. The holes are made about $\frac{1}{20}$ of an inch larger than the diameter of the rivet, in order that the latter may go in easily when heated. Punched holes are generally slightly conical, being larger at the side of the plate farthest removed from the punch. When a great thickness of plates is to be connected, the holes should be drilled; otherwise, the holes in the several plates will be, very likely, not precisely matched, in which case the hole will have to be straightened with a drift pin — an operation which should not be employed, if possible to avoid it.

Machine Riveting.

Machine riveting is rapid, closing up about 12 rivets in the same time that one is closed by hand; it upsets the metal more effectually, and fills the holes without damaging the head, but possesses no marked advantage in point of strength over riveting by hand. Snap riveting has been regarded as a little stronger than that by hand, and is from 50 to 100 per cent. more expeditious,

but it jars the plates more. In hand riveting the heads are often hammered after they are cool, making them weak at the neck. Long rivets require to be cooled at the centre, before inserting, to lessen the shrinkage after the rivet is clinched. Rivets from 6 to 8 inches long, without such a precaution, have often had their heads drawn off, from contraction in cooling.

The shearing strength of steel is about a fourth less than its tensile strength. Mr. Kirkaldy found steel rivets of greater diameter than those used in riveting iron plates of the same thickness to be too small for riveting steel plates.

The Eye-bar and Pin Joint.

One of the most simple and economical joints in use for connecting the parts of structures of iron, and much used in the various forms of truss bridges, is made by means of an eye-bar and pin. This connection may be employed as in the chains of suspension bridges, shown in Fig. 61, or for the junction of

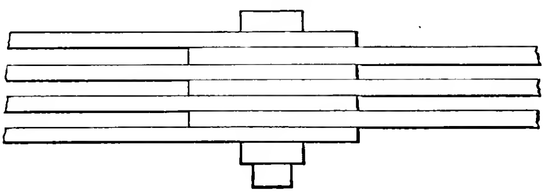


Fig. 61.

the chords and ties of open-work girders, as shown in a following chapter.

By a proper arrangement of the parts of this joint the full strength of the bars may be maintained. To effect this, the collective resistance through E F and E′ F′, Fig. 62, the resistance to the shearing out of the pin through C D, the shearing strength of the pin itself, and the resistance of the same to bending, must each be at least equal to the tensile strength of the bar at A B. In practice the resistances at E F and C D should be more, on account

of the indirectness of the strain at those parts. There is, moreover, an important point to be considered in these joints, which has often been neglected, viz., the diameter of the pin which

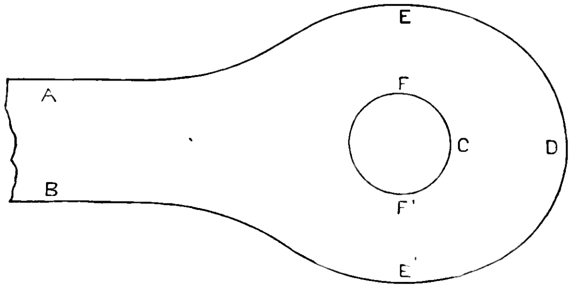


Fig. 62.

shall give a sufficient *bearing area*. Attention was first called to this by Sir Charles Fox, in a valuable paper published in 1865,* in which it is shown that when the pin is too small it upsets the metal at C, elongating the hole, and stretching the metal at F and F′, and finally tearing it from F to E and from F′ to E′. By the experiments of the above gentleman it is shown that increasing the diameter from E to E′ in no way increases the strength of the link, as it does not prevent the crushing at C; but that to obtain the full efficiency of the link, the area of its semi-cylindrical surface bearing on the pin must be a little more than equal to the smallest transverse sectional area of its body — a result to be obtained only by giving a sufficient diameter to the pin. The links experimented upon were 12 feet long, 10¼ inches deep, and one inch thick, the head being 16½ inches diameter, and bored for a 4½ inch pin. The ultimate strength of the bar was 270 tons; but it broke through E F F′ E′ with only 180 tons. Increasing the width (diameter) of the head 2 inches gave no additional strength. Enlarging the pin to 6 inches in diameter, without increasing the width of the head, thus actually diminishing the

* Proc. Roy. Soc. Vol. XIV. 1865. See also Franklin Journal. Vol. L. 1865.

quantity of material in the head, but increasing the bearing of the semi-cylindrical surface from 7 to 9.4 sectional inches, the link required 240 tons to tear it asunder. As a practical conclusion, Mr. Fox recommends that in a 10 inch link the pin should be $6\frac{2}{3}$ inches in diameter, that the sum of the widths of the iron on the two sides of the hole should be 10 per cent. greater than that of the body itself, and that the same proportions be used for other links. It is further suggested that hollow steel pins will save weight, while still giving the required shearing strength.*

As a remarkable violation of the above principles, Mr. Fox refers to a recently erected suspension bridge in which links 10 inches deep have pins only 2 inches in diameter, thus rendering useless two thirds of the metal in the bars.

In proportioning joints of the above description we have first to determine the section of the body of the bars to resist the tension, and then to proportion the head as above.

It will be seen from the preceding remarks that it is very essential to have a good fit between the pin and the eye. The pins should be carefully turned, and the eyes drilled to a gauge. In the bridge across the Rhine, near Mainz, the eyes are ground slightly tapering, and the pin, also tapering, is drawn tightly in by a nut, and the head riveted down.

In making a link or bar like that shown in Fig. 62, especial care is necessary in the manufacture in order not to damage the material. Several methods are employed for this purpose. In England bars have been rolled so as to make the ends wider than the body; this produces an excellent result, but is expensive, and

* In a paper read before the Institution of Civil Engineers, by Mr. George Berkley, in 1870, it was stated that the link of equal strength had been shown by experiment to have the following proportions: —

The body of the link.	100
Diameter of the pin.	74
Depth of head beyond pin.	100
Two sides of the pin hole, ...	125
Radius of curve of neck, ...	150

limited in its application. In the United States the head is produced either by welding a short piece on to the ends of a flat bar, or by upsetting the ends of the bar itself either by hammering or by pressure. Builders are divided upon the relative merits of welding and of upsetting.

The Welding and Upsetting of Eye-bars.

Iron, it is well known, is injured by being brought to a welding heat without being at the same time hammered, as the artificial density caused by the rolling is partially lost where the heat ends : under careful treatment, however, this injury is but little ; and tests, have shown a good scarf weld to be very nearly as strong as the bar itself. The defect in upsetting arises, not from the heating and compressing of the metal where it is upset, but from the change in structure that occurs a short distance back of the head, where it is not much heated, but where the bar is held firmly in order to furnish the necessary reaction to the upsetting tool. The effect at this point is the same as that produced by hammering a piece of cold iron, viz., to change its structure from fibrous to granular, or crystalline. This effect is particularly marked when the upsetting is done by blows from a hammer ; when performed by compression the evil is much less, and if care is taken in the heating, the strength is reduced but very little. A great many tests made upon bars upset by pressure, in the best manner at present known, are stated to have shown the iron just back of the head to have lost in some degree its fibre, in many cases to have become granular. It is difficult to tell from our present knowledge whether bars upset in the most approved manner by pressure are damaged enough to make it any objection to the process.*

* Six letters, three from the most prominent bridge manufacturers, and three from leading engineers in bridge construction, are so entirely contradictory that it is impossible to decide from them whether upsetting does or does not practically injure eye-bars. A letter from one gentleman states plainly

At the Keystone Bridge Works, in Pittsburgh, the upsetting is done by a hammer, the upset being made larger than required, and afterwards reheated and reworked under a steam hammer. At Phœnixville the upsetting is done by pressure, and the subsequent treatment further consolidates the metal in the head. At the Detroit Bridge Works welding is preferred. The work from all these establishments is widely known, and has a deservedly high reputation.

The Screw Joint.

The connection by a nut and screw is simple and economical, the full strength of a rod being obtained by a slight enlargement of the diameter at the end by upsetting; and even without this, though the resistance is somewhat reduced, the arrangement is a good one. The strength of a screw bolt, or a screwed rod, is measured by the sectional area; and the resistance of the screw to being stripped off by the nut depends upon the form of the thread and the depth of the nut. The shearing area in a V screw is equal to the circumference of the bottom of the thread multiplied by the depth of the nut: with square threads the shearing area is only half of the above. Much has been done recently towards introducing a uniform system of screw threads, so that nuts and bolts, wherever found, will be easily and truly matched. The following proportions have been recommended by a committee of the Franklin Institute, and are adopted by the United States Standard Nut Company, of Boston: * —

that the upset bars from a certain shop are entirely uninjured: while another letter, from the engineer of one of the best made iron bridges in America, states as plainly that the bars from the same shop were so defective from the process of upsetting that the ends were all cut off and new ones welded on. Some decided and well arranged experiments to settle this question are very much needed, in which the bars made both by hammering and by pressure should be tested, both for toughness and for strength; not by sudden breaking over an anvil, but by direct tension, particular regard being paid to the behavior of the part of the bar just back of the head.

* Mr. Whitworth's table of screw threads may be seen in the Appendix.

Diameter of Bolt.	Threads per Inch.	Six sided Nut.		Depth of Head.	Depth of Nut.	Strength at 10,000 lbs. per Inch
		Long Diam.	Short Diam.			
$\frac{1}{4}$	20	$\frac{9}{16}$	$\frac{1}{2}$	$\frac{1}{4}$	$\frac{1}{4}$	491
$\frac{5}{16}$	18	$\frac{11}{16}$	$\frac{19}{32}$	$\frac{19}{64}$	$\frac{5}{16}$	767
$\frac{3}{8}$	16	$\frac{25}{32}$	$\frac{11}{16}$	$\frac{11}{32}$	$\frac{3}{8}$	1,104
$\frac{7}{16}$	14	$\frac{43}{48}$	$\frac{25}{32}$	$\frac{25}{64}$	$\frac{7}{16}$	1,503
$\frac{1}{2}$	13	1	$\frac{7}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	1,963
$\frac{9}{16}$	12	$1\frac{7}{64}$	$\frac{31}{32}$	$\frac{31}{64}$	$\frac{9}{16}$	2,485
$\frac{5}{8}$	11	$1\frac{7}{32}$	$1\frac{1}{16}$	$\frac{17}{32}$	$\frac{5}{8}$	3,068
$\frac{3}{4}$	10	$1\frac{7}{16}$	$1\frac{1}{4}$	$\frac{5}{8}$	$\frac{3}{4}$	4,418
$\frac{7}{8}$	9	$1\frac{21}{32}$	$1\frac{7}{16}$	$\frac{23}{32}$	$\frac{7}{8}$	6,013
1	8	$1\frac{7}{8}$	$1\frac{5}{8}$	$\frac{13}{16}$	1	7,854
$1\frac{1}{8}$	7	$2\frac{3}{32}$	$1\frac{13}{16}$	$\frac{29}{32}$	$1\frac{1}{8}$	9,940
$1\frac{1}{4}$	7	$2\frac{5}{16}$	2	1	$1\frac{1}{4}$	12,271
$1\frac{3}{8}$	6	$2\frac{1}{2}$	$2\frac{3}{16}$	$1\frac{3}{32}$	$1\frac{3}{8}$	14,848
$1\frac{1}{2}$	6	$2\frac{3}{4}$	$2\frac{3}{8}$	$1\frac{3}{16}$	$1\frac{1}{2}$	17,671
$1\frac{5}{8}$	$5\frac{1}{2}$	$2\frac{15}{16}$	$2\frac{9}{16}$	$1\frac{9}{32}$	$1\frac{5}{8}$	20,739
$1\frac{3}{4}$	5	$3\frac{3}{16}$	$2\frac{3}{4}$	$1\frac{3}{8}$	$1\frac{3}{4}$	24,052
$1\frac{7}{8}$	5	$3\frac{13}{32}$	$2\frac{15}{16}$	$1\frac{15}{32}$	$1\frac{7}{8}$	27,611
2	$4\frac{1}{2}$	$3\frac{5}{8}$	$3\frac{1}{8}$	$1\frac{9}{16}$	2	31,416
$2\frac{1}{4}$	$4\frac{1}{2}$	$4\frac{1}{16}$	$3\frac{1}{2}$	$1\frac{3}{4}$	$2\frac{1}{4}$	39,760
$2\frac{1}{2}$	4	$4\frac{1}{2}$	$3\frac{7}{8}$	$1\frac{15}{16}$	$2\frac{1}{2}$	49,087
$2\frac{3}{4}$	4	$4\frac{29}{32}$	$4\frac{1}{4}$	$2\frac{1}{8}$	$2\frac{3}{4}$	59,395
3	$3\frac{1}{2}$	$5\frac{3}{8}$	$4\frac{5}{8}$	$2\frac{5}{16}$	3	70,686

It was found by Mr. Kirkaldy that in screwed bolts the breaking strain was greater when old dies were used in their formation than when the dies were new, owing to the iron becoming harder by the greater pressure required in forming the screw thread when the dies were old and blunt, than when new and sharp. He observes, also, that iron bolts case hardened bore a less breaking strain than when wholly iron, owing to the superior tenacity of the small proportion of steel being more than counterbalanced by the greater ductility of the remaining portion of iron.

CHAPTER X.

ON THE STRAINS IN GIRDERS.

Graphic Determination of Strains in Small Bridges.

It is proposed in the following chapter to lay down as plainly as possible the rules for determining the strains in the various forms of trussed frames or open-work girders. The simplest bridge that could be made would consist of a piece of timber or metal placed in a horizontal position across the opening to be spanned. This, however, would be applicable only to short distances. For wider openings the central point of such a girder may be supported by braces from below, as in Fig. 63. In this

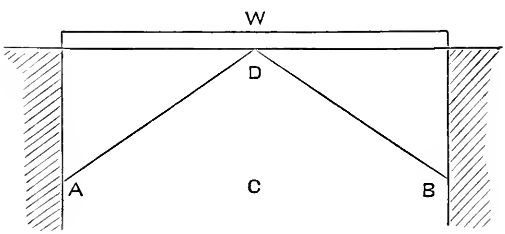


Fig. 63.

case, a load placed at W will produce a thrust on D A or D B equal to half the weight multiplied by the length of the brace and divided by D C, or the vertical reach. The braces will act upon the walls at A and B both vertically and horizontally, and the abutments must be able to sustain this horizontal thrust. If circumstances do not allow the braces beneath the girder, the same result may be accomplished by the arrangement shown in

Fig. 64, in which the centre of the girder A B is suspended from the apex of the braces A D and D B. The strains on these braces

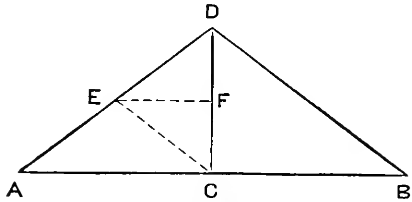


Fig. 64.

are the same as those in Fig. 63. Moreover, the feet of the braces may be applied to the ends of the girder A B, so that the horizontal thrust shall be taken by that beam, and the masonry would have only to support the simple vertical load. This horizontal thrust or tension on A B is found by drawing C E parallel to D B, and E F parallel to A B. If the line D C represents the weight applied at C or D, E F will show the horizontal thrust, or the tension on A B. If the apex of the braces is not over the middle of A B, the load is divided unequally between the braces, and the horizontal thrust is reduced. Thus, in Fig. 65, D C being, as before, the weight, and C E being drawn parallel to B D, D E shows the thrust on the brace A D, D H the thrust on B D, E F the tension, as before. Moreover, D F shows the portion of the load borne by the abutment A, and F C that borne by the abutment B.

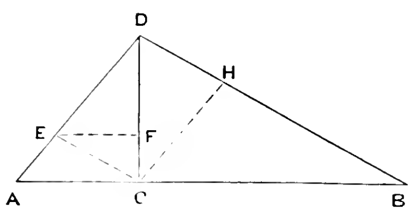


Fig. 65.

The greater the difference between A C and B C, the larger the part of the load borne by A D, and the less the horizontal thrust, until A D becomes vertical, when it bears the whole load, and

there is no horizontal thrust. If the span becomes very long, the braces A D and B D, Fig. 66, will run very high at D, and the unsupported distance from A to C will be too great. We may remedy both of these defects by cutting off the braces at E and F,

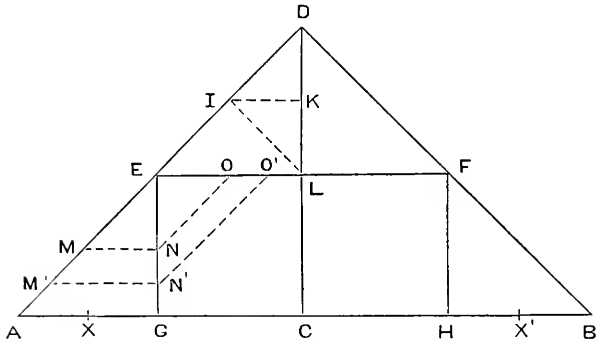


Fig. 66.

and introducing the strut E F to maintain the proper distance between those points, and finally supporting the points G and H from E and F. If the loads applied to E and F were together equal to that applied to D or C, the tension upon A B would be the same whether the braces extended to D or were cut off at E and F, as the angle of the braces is in both cases the same. If D L represents the load applied to C D, then drawing I L parallel to B D, and I K horizontal, the latter shows the tension on A B. So, too, in the second case, if E N, equal to half D L, represents the load applied at G or E, by drawing M N parallel to A B, and N O parallel to A D, we have M N for the tension — evidently equal to I K. Also E O, the same in amount, shows the compression on E F. The load, however, upon E G and F H will be more than that upon C D. The load on C D will be that part of the bridge and load lying between G and H, or one half of the whole, and no more ; the remainder A G and B H being held by the abutments. The load on E G and F H, however, is all that lies between X and X′, or three fourths of the whole ; and laying off half of this latter from E to N′, the corresponding tension is

M′ N′, the compression upon E F is E O′, and the thrust E M′, instead of E M, as before.

If, now, we complete the arrangement shown by A E F B, Fig. 66, by adding the braces G D and H D, and the middle tie C D, in Fig. 67, we have the type of a large class of bridges, viz., all those like the Howe Truss, with inclined braces and vertical ties. The tension thrown by the braces upon the line A B will be the same whatever the angle of the braces. Thus, in the system

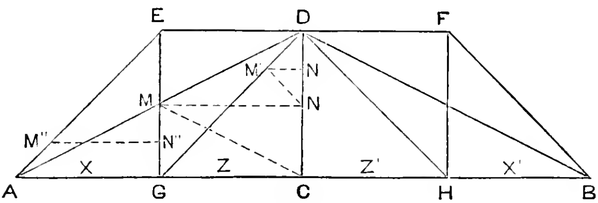


Fig. 67.

A D B, the tie D C sustains the weight from G to H: representing this by the length D C, for convenience one half of G H, and drawing C M parallel to D B, and M N parallel to A B, the latter represents the tension upon A B. Again, in the system A E D F B the central tie D C holds the weight from Z to Z′, represented half size by D N. Drawing N M′ parallel to D H, and M′ N′

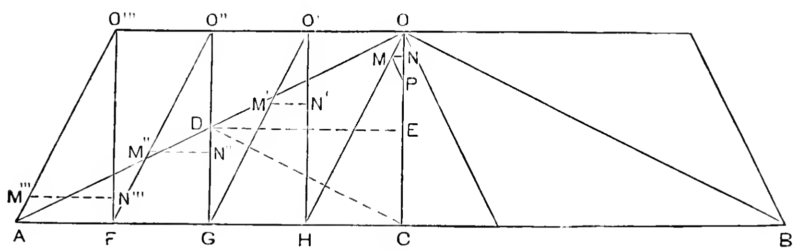


Fig. 68.

parallel to A B, M′ N′ shows the tension on G C from the brace D G. So also making E N″ equal to X C, or otherwise to X Z plus half of Z Z′, and drawing M″ N″ parallel with A B, M″ N″ represents the tension upon A G from the brace A E. The

sum of the tensions M′ N′ and M″ N″ is evidently equal to M N in the system A D B. Again, in Fig. 68, where the braces are inclined only one half as much as in Fig. 67, the sum of the tensions M N, M′ N′, M″ N″, M‴ N‴, will be found equal to the single tension D E from the simple system A O B. It will be seen by this example that the thrust upon the braces increases from the centre to the ends of the truss, each brace transferring its load, by means of the vertical, to the brace next to it: it will

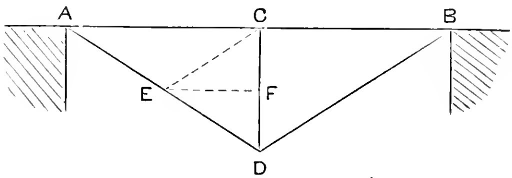


Fig. 69.

also be seen that the tension upon A B increases in the opposite direction, being M‴ N‴ in the part A F, M‴ N‴ + M″ N″ in F G, and so on to the centre, where it is a maximum, and equal to D E. In the same way the compression on the top chord increases from the end to the centre. Where, however, there is

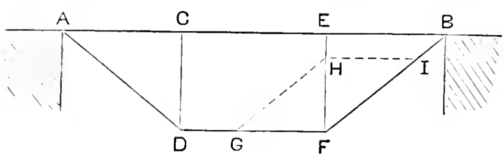


Fig. 70.

but a single pair of braces, as in the system A O B, the tension is the same throughout the line A B. In the above Figs. 64 to 68, the braces or compression members are inclined, while the tension members are vertical; and this form, as before remarked, represents a large class of bridges. If we invert the above several Figs. we obtain a system in which the compression members are vertical, and the tension members are inclined, but inclined in the opposite direction from the braces in the first system. This form also represents a large class of bridges in use for railway

purposes. The manner of estimating the strains is the same as in the system above considered. Thus, if in Fig. 69, C D represents the load on the point C, and we draw C E parallel to B D, and E F parallel to A B, the latter represents the compression on A B, and E D the tension on A D. In Fig. 70, F H representing the weight upon E F, H I is the compression upon A B, and G F = H I the tension on D F, and F I the tension on B F. Again, in Fig. 71, the load on C D being represented by

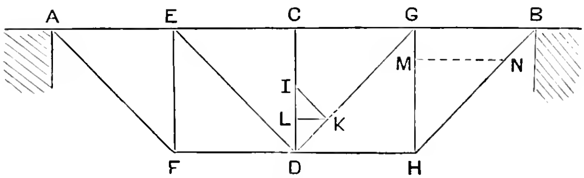


Fig. 71.

I D, the compression on C G is shown by L K ; and the weight on G H being represented by H M, the compression on G B is shown by M N ; and the sum of L K and M N is the total compression at the middle of A B. As in the former system, the compression on A B increases from the end to the centre, while

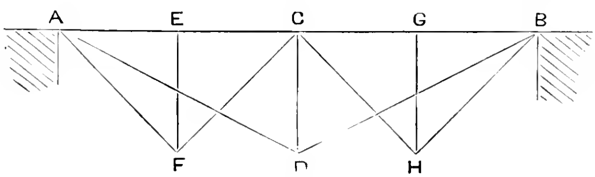


Fig. 72.

the strain upon the ties increases from the centre to the ends. A modification of this system consists in supporting intermediate points as E, C, G, Fig. 72. Here we have a primary system, A D B, and from the point C, thus supported, we establish the secondary systems, A F C and C H B. Other intermediate points may be supported by an extension of the same principle. Another modification of the suspension system is shown in Fig. 73, in which the points E C G are supported upon F D H, the latter being referred

directly to the abutments. In Figs. 72 and 73 the compression upon the upper member A B is uniform from end to end, as already remarked in the case of the system A O B, Fig. 68.

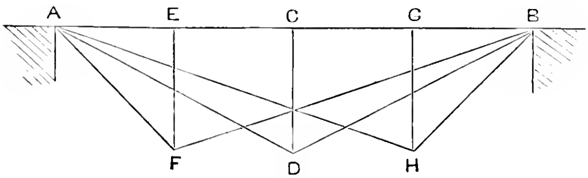


Fig. 73.

Another plan, combining compression and tension members both inclined, is shown in Fig. 74, where A D, D E, and E B are compressed, and A B, C D, and C E are extended. If we make

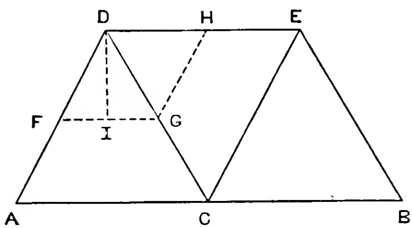


Fig. 74.

D I equal to half the load at C, and draw F G parallel to A B, and G H parallel to A D, then D G shows the tension on D C, D F the compression on A D, and D H that on D E. The ten-

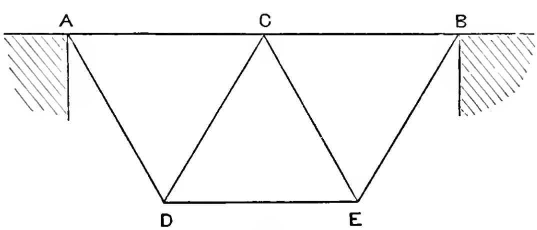


Fig. 75.

sion upon A C, however, is not equal to D H, but is only half as much, as will be seen by resolving the thrust D F into vertical

and horizontal components at A. Inverting Fig. 74, we have Fig. 75, in which A D, D E, and E B are in tension, and C D, C E, and A B in compression. Extending the last two Figs. we have the Warren Girder, or Triangular Truss.

Special Forms of Bridge Trusses.

The preceding forms by modifications, dependent upon the requirements of special cases, furnish nearly all of the open-work girders, or truss bridges, now in use upon railways. The full lines in Fig. 76 show the chords, braces, and ties of the well

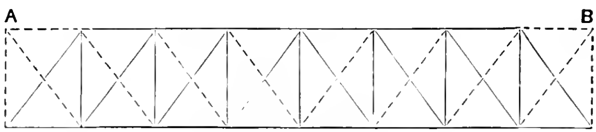


Fig. 76.

known Howe Truss. The dotted diagonals represent the counterbraces in that truss, the use of which will be referred to in advance. This, it will be seen, is essentially the same as the plan

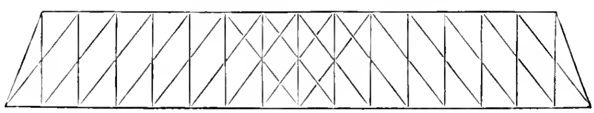


Fig. 77.

shown in Fig. 68. In practice, the upper chords are commonly carried out, as in the Fig., to A and B, and posts introduced at the ends. (See Plate VI.) In this bridge the braces are inclined *up*

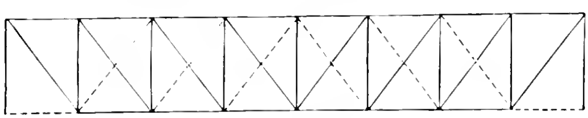


Fig. 78.

towards the centre, and the tension rods are vertical. Fig. 77 is a modification of Fig. 76, made simply by applying an interme-

diate system of braces and ties of the same kind, the chords being common to both systems. Fig. 78 shows the Pratt, or Whipple Bridge, in its simplest form, the compression members

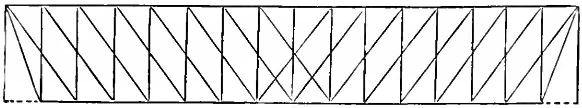


Fig. 79.

being vertical, and the tension members inclined *down towards the centre*. Fig. 79 is a modification of the same, made by introducing an intermediate system. Fig. 80 is another modification

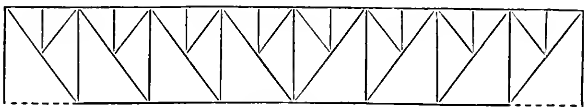


Fig. 80.

of the same, where it is found necessary to support the roadway at frequent intervals. Fig. 81 shows the Warren Girder, the roadway being upon the lower chord. Intermediate points, as

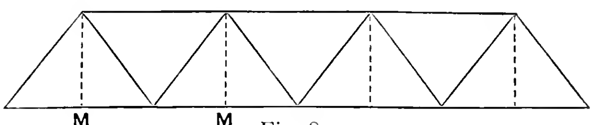


Fig. 81.

M M, may be suspended by rods from the apices of the braces. Fig. 82 shows the same form with the roadway upon the top, in

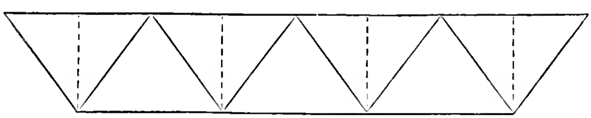


Fig. 82.

which intermediate points may be supported by columns from the apices below. In Fig. 81 the first inclined member is a

brace, in Fig. 82 it is a tie. By combining Figs. 81 and 82 we have the double Triangular, or single lattice, shown in the left hand half of Fig. 83. The right hand half of the same Fig.

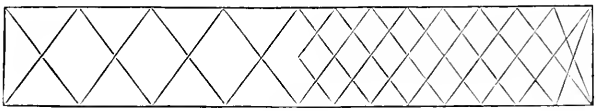


Fig. 83.

shows the same truss, with the addition of an intermediate frame, making a closer lattice, the chords, of course, being the same for the several systems. Fig. 84 shows another modification of the

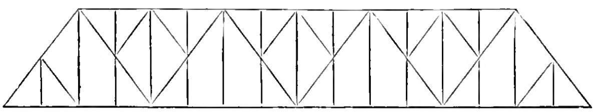


Fig. 84.

Warren Girder, applicable to large spans, where several intermediate points require support. Fig. 85 shows Post's Bridge, the

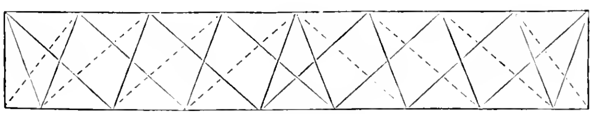


Fig. 85.

compression members being inclined at what the inventor considers the most economical angle: the tension members are

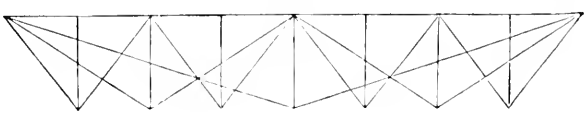


Fig. 86.

placed at a much less inclination. Fig. 86 represents Fink's Suspension Bridge, a form very widely adopted, and fully illus-

trated in advance. Fig. 87 shows Bollman's Bridge, also illustrated in a following chapter.

The strains upon the several parts of these trusses may all be obtained by graphic methods, like those referred to above; but

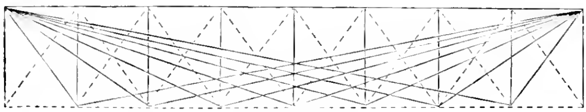


Fig. 87.

in large structures, with many members, the easiest mode of proceeding is that given below, where an example of each kind is worked out. In the following illustrations the dimensions of the chords, ties, and braces will first be obtained upon the supposition that the bridge is loaded with a uniformly distributed and permanent load. Afterwards the effect of a moving load will be considered, and the subject of counterbracing examined.

Calculation of the actual Strains in Trusses.—Fink's Bridge.

Let the span in each case be assumed as 200 feet, the depth of truss 20 feet, the panels $12\frac{1}{2}$ feet each, or 16 in number, and the load 3600 pounds per running foot, or 45,000 pounds per panel, or 22,500 pounds on each side truss. Taking first the Fink Bridge, as one of the simplest, and arranging the parts as in Fig. 88, we proceed as follows: The total load being 3600

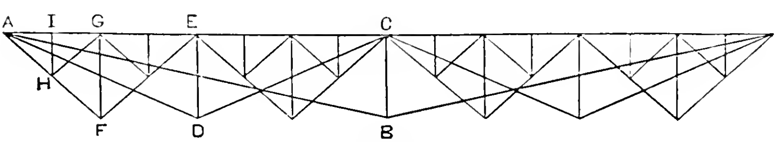


Fig. 88.

pounds per foot, or 1800 pounds per foot on each side truss, the whole weight is 1800×200, or 360,000 pounds. One half of this, or 180,000 pounds, comes on C B; 90,000 on D E; 45,000

on G F, and 22,500 on I H. These are the simple vertical weights applied to each pair of tension bars, one half of which is borne by each bar ; but on account of their inclination the actual strain is increased as below.

	Load on the Post.	Vertical Load on each Bar.	Length of the Bar.	Angle of Bar or Multiplier.	Strain on the Bar.
C B	180,000	90,000	$\sqrt{100^2+20^2}=102$	$\frac{102}{20}$	459,000 A B
E D	90,000	45,000	$\sqrt{50^2+20^2}=54$	$\frac{54}{20}$	121,500 C D
G F	45,000	22,500	$\sqrt{25^2+20^2}=32$	$\frac{32}{20}$	36,000 E F
I H	22,500	11,250	$\sqrt{12\frac{1}{2}^2+10^2}=16$	$\frac{16}{10}$	18,000 G H

Each of the simple systems, A B, A D C, A F E, etc., throws a compression upon the top chord, found as below.

$$
\begin{array}{llcrcr}
\text{A B} & \frac{100}{20} & \times & 90,000 & = & 450,000 \\
\text{C D} & \frac{50}{20} & \times & 45,000 & = & 112,500 \\
\text{E F} & \frac{25}{20} & \times & 22,500 & = & 28,125 \\
\text{G H} & \frac{12\frac{1}{2}}{10} & \times & 11,250 & = & 14,062 \\
\end{array}
$$

Total compression, 604,687 pounds.

Bollman's Bridge.

In the Bollman Bridge, Fig. 89 (the central and end suspension systems only are drawn, to save confusion of lines), each

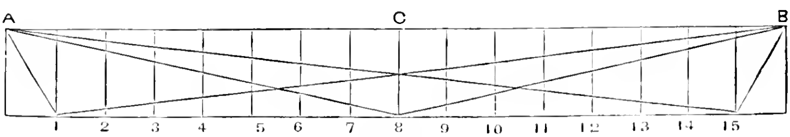


Fig. 89.

pair of bars, as A 1 and B 1, is independent, and supports the load on one post. The load on the posts is also the same throughout, and is the weight of one panel, or 22,500 pounds for each side truss. In the system A 1 B, the bar A 1 bears $\frac{15}{16}$, and B 1 $\frac{1}{16}$ of the load on the post : in the system A 2 B, A 2 bears

$\frac{14}{16}$, and B 2 $\frac{2}{16}$ of the load on the post, and so on for the other systems. Dividing the load in these proportions, and multiplying the vertical weight at the foot of each bar by the angle of the bar, we get the following table, the last column of which shows the direct tensile strain on the different members :—

No. of the bar	Length of the Bar.	Weight at Foot of Post.	Tension on the Bar, or Weight multiplied by the Length of Bar and divided by the Depth of Truss.
A 1	$\sqrt{12\frac{1}{2}^2 + 20^2} = 23.6$	$22{,}500 \times \frac{15}{16} = 21{,}094$	$21{,}094 \times \frac{23.6}{20} = 24{,}890$
A 2	$\sqrt{25^2 + 20^2} = 32.0$	$22{,}500 \times \frac{14}{16} = 19{,}687$	$19{,}687 \times \frac{32}{20} = 31{,}499$
A 3	$\sqrt{37\frac{1}{2}^2 + 20^2} = 42.5$	$22{,}500 \times \frac{13}{16} = 18{,}281$	$18{,}281 \times \frac{42.5}{20} = 38{,}847$
A 4	$\sqrt{50^2 + 20^2} = 53.9$	$22{,}500 \times \frac{12}{16} = 16{,}875$	$16{,}875 \times \frac{53.9}{20} = 45{,}478$
A 5	$\sqrt{62\frac{1}{2}^2 + 20^2} = 65.6$	$22{,}500 \times \frac{11}{16} = 15{,}469$	$15{,}469 \times \frac{65.6}{20} = 50{,}738$
A 6	$\sqrt{75^2 + 20^2} = 77.6$	$22.500 \times \frac{10}{16} = 14{,}062$	$14{,}062 \times \frac{77.6}{20} = 54.560$
A 7	$\sqrt{87\frac{1}{2}^2 + 20^2} = 89.8$	$22{,}500 \times \frac{9}{16} = 12{,}656$	$12{,}656 \times \frac{89.8}{20} = 56{,}825$
A 8	$\sqrt{100^2 + 20^2} = 102.0$	$22{,}500 \times \frac{8}{16} = 11{,}250$	$11{,}250 \times \frac{102}{20} = 57{,}375$
A 9	$\sqrt{112\frac{1}{2}^2 + 20^2} = 114.0$	$22.500 \times \frac{7}{16} = 9{,}844$	$9{,}844 \times \frac{114}{20} = 56{,}111$
A 10	$\sqrt{125^2 + 20^2} = 126.6$	$22{,}500 \times \frac{6}{16} = 8{,}437$	$8{,}437 \times \frac{126.6}{20} = 53{,}406$
A 11	$\sqrt{137\frac{1}{2}^2 + 20^2} = 138.9$	$22.500 \times \frac{5}{16} = 7{,}031$	$7.031 \times \frac{138.9}{20} = 48{,}830$
A 12	$\sqrt{150^2 + 20^2} = 151.3$	$22{,}500 \times \frac{4}{16} = 5{,}625$	$5{,}625 \times \frac{151.3}{20} = 42{,}553$
A 13	$\sqrt{162\frac{1}{2}^2 + 20^2} = 163.7$	$22{,}500 \times \frac{3}{16} = 4{,}219$	$4{,}219 \times \frac{163.7}{20} = 34.532$
A 14	$\sqrt{175^2 + 20^2} = 176.1$	$22{,}500 \times \frac{2}{16} = 2{,}812$	$2{,}812 \times \frac{176.1}{20} = 24{,}759$
A 15	$\sqrt{187\frac{1}{2}^2 + 20^2} = 188.6$	$22{,}500 \times \frac{1}{16} = 1{,}406$	$1{,}406 \times \frac{188.6}{20} = 13{,}258$

The other set of bars, B 15, B 14, etc., are, of course, strained to the same amount. The compression thrown upon the top chord by any system is found by multiplying the direct strain upon any bar by the distance from the abutment to the top of the post connected with that system, and dividing the product by the length of the bar. Thus, —

System	Tension.	Multiplier.	Compression.
A 1	24,890	$\frac{12\frac{1}{2}}{23 \cdot 6}$	13,183
A 2	31,499	$\frac{25}{32}$	24,608
A 3	38,847	$\frac{37\frac{1}{2}}{42 \cdot 5}$	34,277
A 4	45,478	$\frac{50}{53 \cdot 9}$	42,187
A 5	50,738	$\frac{62\frac{1}{2}}{65 \cdot 6}$	48,340
A 6	54,560	$\frac{75}{77 \cdot 6}$	52,732
A 7	56,825	$\frac{87\frac{1}{2}}{89 \cdot 8}$	55,370
A 8	57,375	$\frac{100}{102 \cdot 0}$	56,250

To find the whole compression upon the upper chord, we double the sum of the first seven systems above, and add thereto the eighth, making the whole 597,644 pounds.

The Whipple or Pratt Bridges.

The two trusses above considered have no lower chords, and the compression upon the top chords is uniform from end to end. In the trusses examined below we have a lower chord under tension; and the compression upon the upper chord and the tension upon the lower one decreases, panel by panel, from the centre to the ends.

Let Fig. 90 represent the simplest form of the Pratt or Whipple Bridge, A B, C D, E F, etc., being the tension bars, and the ver-

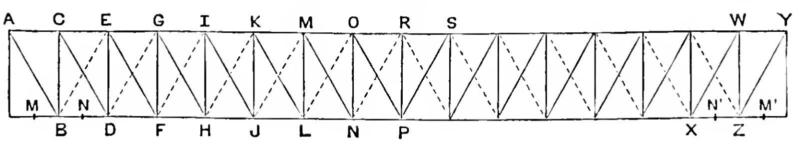


Fig. 90.

ticals the compression members. In this bridge the end ties, A B and Y Z, have evidently to support all the load lying between the middles of the end panels, or M and M′: the ties C D and W X have to sustain all between N and N′: or, in other words, each tie has to hold the weight of the bridge and load between the centre

of the bridge and the middle of the panel back of it, and this weight must be multiplied by the length of the tie and the product divided by the length of post to obtain the direct strain on the tie. Thus we have, —

$$\text{Weight} \times \frac{\text{length of tie}}{\text{length of post}} = \text{strain on tie,}$$

$$\text{Weight} \times \frac{\text{length of panel}}{\text{length of post}} = \text{compression on chord.}$$

As long as all the ties have the same angle, we find the above ratios once for all, and use them as multipliers, as in the table following. The post being 20 feet high, and the panel $12\frac{1}{2}$ feet, the diagonal is 23.6. The multiplier for the strain on the ties is thus $\frac{23.6}{20}$, or 1.18, and that for the compression on the chords is $\frac{12\frac{1}{2}}{20}$, or .625.

Weight at Foot of the Bar.	Tension on Tie = W × 1.18.	Compression on Chords = W × .625	Accumulated Compression on Panels
A B $7\frac{1}{2}$ P or 168,750	199,125	105,469	A C 105,469
C D $6\frac{1}{2}$ P " 146,250	172,575	91,406	C E 196,875
E F $5\frac{1}{2}$ P " 123,750	146,025	77,344	E G 274,219
G H $4\frac{1}{2}$ P " 101,250	119,475	63,281	G I 337,500
I J $3\frac{1}{2}$ P " 78,750	92,925	49,219	I K 386,719
K L $2\frac{1}{2}$ P " 56,250	66,375	35,156	K M 421,875
M N $1\frac{1}{2}$ P " 33,750	39,825	21,094	M O 442,969
O P $\frac{1}{2}$ P " 11,250	13,275	7,031	O R 450,000

The tension in the panel B D of the lower chord is the same as the compression in the panel A C of the upper chord ; or the compression and tension upon the same side of any brace are equal, until we reach the centre. In the first panels of the lower chord, M and M′, there is no tension ; and no chord is needed in

that panel, unless for connection with the masonry or for support of the roadway.

In the case of the Howe Bridge we have only to invert Fig. 90, when the ties become braces, and the above compressions on the top chord become the tensions on the lower chord, and the tensions on the lower chord become compressions on the top chord, in which case there is no compression in the end panels of the upper chord; and the end section of the chord, and the end post, may be omitted, unless required for lateral bracing.

The Warren or Triangular Girder.

Let Fig. 91 represent a Warren Girder, with the roadway upon the lower chord. The panels here are twice as long, and thus the load applied at the feet of the ties is double that in the pre-

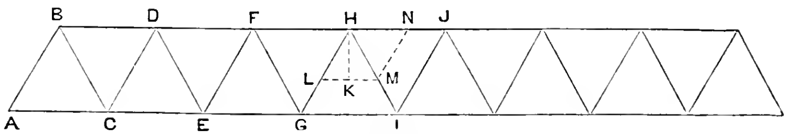


Fig. 91.

ceding cases, or 45,000 pounds. Moreover, the strain changes its character at each apex; being first compression, and then tension; A B, C D, etc., being braces, and B C, D E, etc., ties.

Weight at Foot of Tie, or Top of Brace.	Resulting Strain on Tie or Brace.	Compression on Top Chord w × .625 × 2.	Accumulated Compression.	Tension on lower Chord.	Accumulated Tension.
I or H 22.500	G H or H I 26.550	H J 28.125	450.000	G I 56.250	435.937
G or F 67.500	E F or F G 79.650	F H 84.375	421.875	E G 112.500	379.687
E or D 112.500	C D or D E 132.750	D F 140.625	337.500	C E 168.750	267.187
C or B 157.500	A B or B C 185.850	B D 196.875	196.875	A C 98.437	98.437

Col. 1 gives the load at the foot of each tie, which is transferred at once to the top of the brace behind it: the load at I is the weight of one panel, one half of which is taken by H I, and

one half by J I. The load at G, is the half panel thrown upon it by the brace H G, and the two half panels adjacent to G; or in all one and a half panels. In the same way the load at E is $2\frac{1}{2}$, and at C $3\frac{1}{2}$ panels. The direct strain upon any brace, as shown in col. 2, is found by multiplying the weight in col. 1 by the length of the tie, and dividing by the length of the vertical. The compressions in col. 3 are obtained by multiplying the weights in col. 1, by .625, and by 2, or by double the ratio .625 used in the preceding example. The reason for this will be seen by reference to the resolution of forces upon the line I H, where, calling H K the vertical load upon H, H M will be the tension on the tie H I, and L M or H N the compression on H J. The angle between a brace and tie being twice as great as in the preceding example, L M, or the horizontal component of H M, is twice as much.

The tension on the lower chord in the above example is not, as in the preceding case, the same as the compression in the top chord on the same side of any given diagonal, but it follows a different ratio of increase in the top and bottom chords, from panel to panel, according to the arrangement of the members of the truss and the position of the load. Thus, in Fig. 92, where

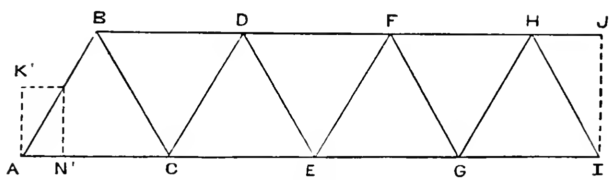


Fig. 92.

A B is a brace, and the load is on the bottom, the weights borne by the respective members are $\frac{1}{2}$ panel on H I and H G, $1\frac{1}{2}$ on F G and F E, $2\frac{1}{2}$ on D E and D C, and $3\frac{1}{2}$ on B C and B A. The compression in H J is that due to $\frac{1}{2}$ a panel, that in F H to $1\frac{1}{2}$ panels, in D F to $2\frac{1}{2}$ panels, and B D to $3\frac{1}{2}$ panels. On the lower chord, however, the tension on G I is that due to 1 panel, that on E G to 2 panels, and that on C E to 3 panels. In Fig. 93,

where A B is a tie, and the load is on the top, the strain upon the upper panel, G I, is that due to 1 panel, that on E G, to 2 panels,

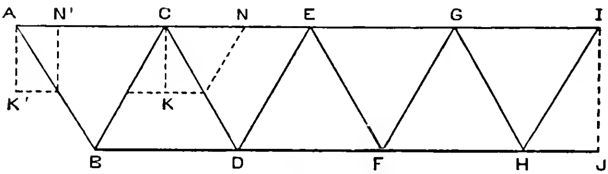


Fig. 93

that on C E to 3 panels: while the strain on H J of the lower chord is that due to $\frac{1}{2}$ a panel, on F H to $1\frac{1}{2}$, on D F to $2\frac{1}{2}$, and on B D to $3\frac{1}{2}$ panels. Reducing the above to whole numbers, we have the following ratios:—

In Fig. 92,	Top,	H J = 1.	F H = 3.	D F = 5.	B D = 7.	
	Bottom,	G I = 2.	E G = 4.	C E = 6.	A C = $3\frac{1}{2}$.	
In Fig. 93,	Top,	G I = 2.	E G = 4.	C E = 6.	A C = $3\frac{1}{2}$.	
	Bottom,	H J = 1.	F H = 3.	D F = 5.	B D = 7.	

Having found the actual tension or compression upon H J, we have only to multiply it by the above numbers in the two different cases to get the strains in the other panels, and adding the several strains from the ends to the centre we get the accumulated panel strains. The former example Fig. 91, will serve to illustrate the process. It will be seen that the strain in the end panel, A C, Figs. 92 and 93, has for its multiplier only $3\frac{1}{2}$, or one half as much as the panel B D. This will be understood by observing the resolution of forces at C and at A, Fig. 93, in which making C K and A K′ the load upon the apices C and A, the horizontal component at A N′ is seen to be one half of that at C N. The thrust of the tie A B, in Fig. 93, or of the brace, in Fig. 92, is resolved at the point A into a horizontal component along the chord, and a vertical component resisted by the upward reaction of the abutment.

STRAINS DUE TO A MOVING LOAD.—COUNTERBRACING.

In the previous examples the strains upon the parts of the several trusses have been estimated upon the assumption that the load was uniform and permanent. This, however, is not the case with railway bridges. The trains which cross them are brought upon them in a partial, sudden, and somewhat violent manner; and to provide for this, an allowance must be made in proportioning the parts. If a load is placed at C, Fig. 94, $\frac{7}{8}$ of it must go to the

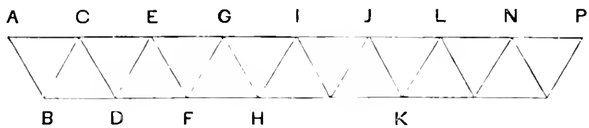


Fig. 94.

abutment A, and $\frac{1}{8}$ of it to the abutment P. So, too, if a load is placed at E, $\frac{6}{8}$ of it must go to A, and $\frac{2}{8}$ of it to P. If a train of uniform density covered $\frac{1}{2}$ of the bridge, from A to I, the centre of gravity of that load would be over the point E, and $\frac{3}{4}$ of the weight would be carried to A, and $\frac{1}{4}$ of it to P. Now this one fourth can only get from the point E to P when the members E F and G H act by compression, and the members G F and H I act by tension; strains just the reverse of those which these members are called upon to resist under a uniform and permanent load. Moreover, a member like E F may be called upon suddenly, as the train moves on to the bridge, to change its action from resisting compression to resisting tension, or the reverse. Besides this, each tie has to bear a large part of the weight of the engine as it passes. While, however, the effect of a moving load is to increase the strain upon the members of the unloaded part of the bridge, it has precisely the reverse effect upon those of the loaded portion. As the advancing train passes from A to I, the strains upon the main ties, A B, C D, E F, and G H, are diminished, or even reversed, since that portion of the load to be carried from C, E, or G to the unloaded abutment can only be trans-

ferred to the centre when the ties C D, E F, etc., act as braces, and the members E D, G F, etc., as ties. The maximum direct strain in any brace, as L K, Fig. 94, occurs when the train is moving from A to P, and the engine is at J; as the brace in question then supports its share of the bridge itself, a large part of the concentrated weight of the engine, and the portion of the load to be transferred to the unloaded abutment.* The minimum direct, or maximum reverse, strain occurs in the same brace when the train is moving in the opposite direction, and the engine is at L. So long as the reverse strain thrown by the passage of a train upon any member is less than the direct strain, the proportion of the load to be transferred to the unloaded abutment will be provided for; but when the reverse strain becomes greater than the direct strain, the member must be so arranged as to resist the reverse strain, or else a new member, termed a counterbrace, must be introduced for that purpose. The counterbraces, as shown in several of the Plates, are always opposed in position or in action to the main braces. If they occupy the same diagonal as the main brace, they act in the opposite manner; i. e., by tension when the main brace acts by compression, and by compression when the main brace acts by tension. If they occupy the opposite diagonal, they act in the same manner; i. e., by tension when the main brace acts by tension, and by compression when the main brace acts by compression. In transferring a portion of the load to the unloaded abutment, C D, Fig. 94, will be required to carry $\frac{1}{5}$ of the load at C, and E F will be required to carry $\frac{2}{5}$ of the load at E. If the points C, E, and G are all loaded, C D will carry $\frac{1}{5}$ of the load at C, E F will carry the $\frac{1}{5}$ received from C D, added to its own $\frac{2}{5}$, and

* "The maximum tensile strain in any diagonal occurs when the passing train covers the segment *from which* the diagonal slopes upward; and the maximum compressive strain when it covers the segment *towards which* the diagonal slopes upward." — *Stoney*, Vol. I. p. 97.

G H will in the same way carry $\frac{1}{8} + \frac{2}{8} + \frac{3}{8}$, or $\frac{6}{8}$ of the load placed upon one apex.

The permanent load upon the braces at the end of a long truss is always greater than the proportion of the moving load to be transferred to the unloaded abutment, and thus counterbraces are theoretically unnecessary at that part of the bridge. Towards the centre, however, the permanent load becomes much smaller, as seen in the examples given above, and thus the moving load is much larger in proportion, and counterbraces are needed. In small bridges the permanent load is not enough to overbalance the moving load, and counterbraces may be needed throughout. To ascertain whether the strains upon any member are direct or the reverse, we have only to suppose the bridge to be loaded at the several points, and to estimate the separate strains upon the different members, the algebraic sum of which will show the nature of the strain to be provided for. For example, let the moving load upon the truss, Fig. 94, be 8 tons at each apex. Of the 8 tons at C, 7 go to the abutment A, and one to the abutment P. Of the 8 tons at E, 6 go to A, and 2 to P, and so on throughout the truss. The one ton that goes from C to P produces a compression, or a reverse strain, upon C D, a tension on D E, a compression on E F, and so on, to the centre. So, too, in the case of the load at E, the two tons that go from E to P produce a compression on the tie E F, a tension on F G, and so on.

	C	E	G	I	J	L	N	8	9	10	11	12
A B	− 7	− 6	− 5	− 4	− 3	− 2	− 1	− 28	+ 0	− 14	− 42	+ 0
B C	+ 7	+ 6	+ 5	+ 4	+ 3	+ 2	+ 1	− 0	+ 28	+ 14	− 0	+ 42
C D	+ 1	− 6	− 5	− 4	− 3	− 2	− 1	− 21	+ 1	− 10	− 31	+ 0
D E	− 1	+ 6	+ 5	+ 4	+ 3	+ 2	+ 1	− 1	+ 21	+ 10	− 0	+ 31
E F	+ 1	+ 2	− 5	− 4	− 3	− 2	− 1	− 15	+ 3	− 6	− 21	+ 0
F G	− 1	− 2	+ 5	+ 4	+ 3	+ 2	+ 1	− 3	+ 15	+ 6	− 0	+ 21
G H	+ 1	+ 2	+ 3	− 4	− 3	− 2	− 1	− 10	+ 6	− 2	− 12	+ 4
H I	− 1	− 2	− 3	+ 4	+ 3	+ 2	+ 1	− 6	+ 10	+ 2	− 4	+ 12

In the above table each weight is divided according to its position between the abutments, and the effect upon each member.

both in amount and character is stated, + denoting compression, and — tension. No allowance, however, is made for the increase of strain due to the inclination of the members, the object being merely to show the general character and amount of the forces.

Col. 8 shows the sum of the — values in the seven preceeding columns, and col. 9, the + values of the same. Col. 10 gives the amount and character of the strain upon the several members from the permanent load, assumed at 4 tons upon each apex. Col. 11 gives the maximum negative strains, obtained by adding cols. 8 and 10. Col. 12 gives the maximum positive strains, obtained by adding cols. 9 and 10. It will be seen by the table that the members G H and H I are subject to both positive and negative strains. These members therefore require to be made so as to resist both tension and compression, or else an additional member, the counterbrace, must be introduced. Methods of meeting the first requirement are shown in Plate XII, and also in Plate XIII; while the ordinary counterbraces are shown in Plates VI, VII, VIII, X, and XI.

It was formerly held by engineers that the use of counterbracing was simply to throw a permanent load upon the main braces, or to maintain a portion of the depression produced in the bridge by the passage of a load after the load was removed. It is easy to see that if we depress a bridge by loading it, and anchor it firmly to the ground, it would not be able to recover its form when the load was removed; but would remain under the same strain as when loaded, though from a different cause, and its rising and falling under subsequent applications of the same weight would be prevented. The same result may be accomplished, to some extent, by counterbracing. Recent authorities, however, do not attach much importance to this operation.* The former practice, especially in wooden bridges, was to counterbrace throughout the whole length of the bridge. At present engineers are divided upon this point; some counterbracing throughout, except in the

* Haupt, pp. 82–84. Whipple, pp. 187–197. Stoney, Vol. II. p. 338.

panels next the abutments, others putting in counterbraces for the few panels only near the centre, as indicated by the reverse strains in the table on page 201, while in many large bridges recently built in Europe for railway purposes no counterbracing is employed. Very much depends upon the details of construction. If the joints are riveted, and the truss is well braced laterally and transversely, but little counterbracing will be required. In a girder with loose joints, and an upper chord formed of many members, not connected so as to have much transverse strength at the joints, counterbraces will be essential throughout. A well framed and stiff floor, by which a concentrated load may be distributed to several panels, is of the first importance. The forces arising from the momentum of cars and engines, swaying up and down upon their springs, are not subject to calculation.

Bowstring Girders.

In what has preceded, the chords of the bridge are supposed to be parallel and horizontal, Another class of girders employed for railway bridges, and shown in Figs. 95 to 100, have one chord

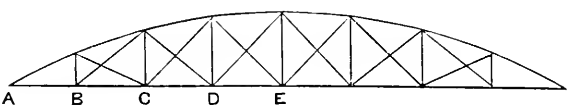


Fig 95.

curved. Fig. 95 represents the simplest form of what is termed a bowstring girder ; the top chord being curved so as to become an arch, with its feet resting upon the lower one. The curved chord of course suffers compression, and the straight or lower one extension. If the load was stationary, it might be suspended at B C D E by simple vertical rods from the arch. Such an arrangement, however, would be entirely unsuited for a moving load ; as the arch would lack the stiffness necessary to preserve its form. This stiffness is obtained by the introduction of the

diagonals shown in Fig. 95 Numerous examples of this form of bridge may be seen in England. Another mode of arranging

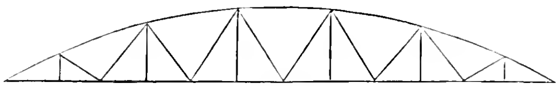


Fig. 96.

the bracing is shown in Fig. 96, where a single set of triangles are introduced, intermediate points being suspended from the apices of the braces by vertical rods. In Fig. 97 a double triangular or

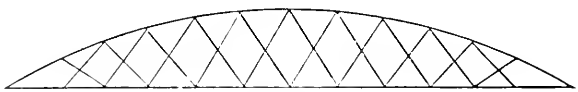


Fig 97.

lattice web is introduced for the connection of the chords. Fig. 98 represents an inverted bowstring girder ; the horizontal chord being subject to compression, and the curved one to extension.

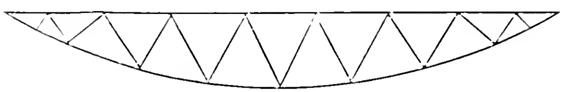


Fig. 98.

This is the general form of a fine steel bridge of 137 feet span, over a rapid torrent upon the Herljunga and Wernersborg Railway in Sweden. Fig. 99 combines the direct and the inverted

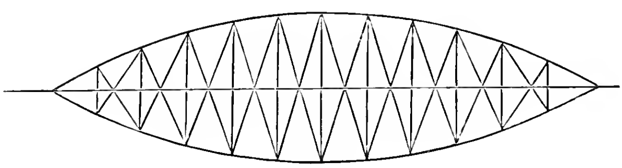


Fig. 99.

bowstrings ; an arrangement illustrated by the Royal Albert Bridge, erected by Mr. Brunel over the Tamar, at Saltash, in

Cornwall. This immense structure has spans of 445 feet; the upper chord being composed of wrought iron cylindrical tubes, and the lower one of wrought iron tension bars. Fig. 100 represents an arch stiffened by spandrel bracing; a form widely adopted in England. Should it be desirable, one roadway may be placed above the arch, and another may be suspended, as shown by the lower horizontal line in the figure. Fig. 101 shows a plan for a

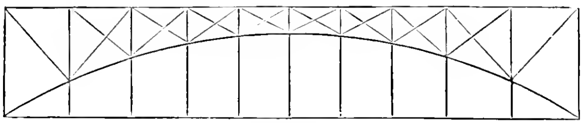


Fig. 100.

larger structure, consisting of a bent girder, upon the back of which the roadway may be supported, either by simple verticals,

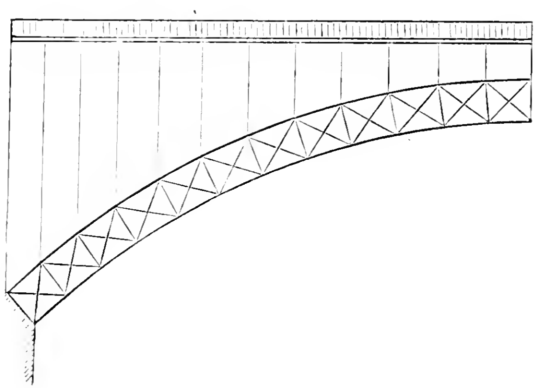


Fig. 101.

as in the sketch, or by verticals stiffened by radial members running in an inclined direction from the back of the arch. Fig. 102 shows the general plan of the great bridge now being built across the Mississippi at St. Louis. The chords of the curved ribs are to be made of cast steel tubes; the thrust of the arches being thrown upon the masonry. This immense work is to consist of

two spans of 497 feet each, and one span of 515 feet, besides the approaches. The common traffic is to pass upon the upper part,

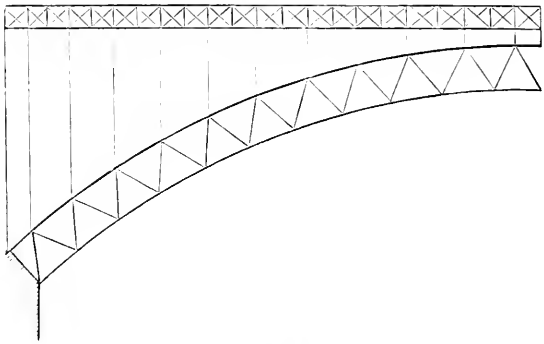


Fig. 102.

the railway being partly supported by and partly suspended from the arch beneath.

The arches in Figs. 101 and 102 may be made deep enough and so braced as to possess sufficient stiffness in themselves to enable them to maintain their form under a passing load without the aid of spandrel bracing. This depth in the St. Louis Bridge is 12 feet.

In the ordinary Bowstring Girder, as shown in Figs. 95, 96, or 97, the strain upon the chords is easily found with a sufficient degree of accuracy for practice, as follows: When the load is uniform, the compression at the centre of the top chord, and the tension throughout the lower one, is given by the formula —

$$S = \frac{W\,l^2}{8\,d};$$

in which S is the total tension or compression in tons, W the load per lineal foot, l the span, and d the versed sine of the arch. Thus the compression at the centre of the arch, the span being 100 feet, the rise 10 feet, and the load one ton per foot, is —

$$S = \frac{1 \times 100 \times 100}{8 \times 10} = 125 \text{ tons.}$$

The compression in the top chord does not decrease from the centre to the end, as in the parallel chords before referred to, but remains nearly the same throughout, increasing somewhat towards the abutment, since the chord, as it becomes more and more inclined, assumes more and more the office of the braces in addition to its own office of a chord. The compression at any point other than the centre is given by the formula —

$$S = \sqrt{\left(\frac{W\ l^2}{8\ d}\right)^2 + (W\ y)^2};$$

where y is the distance from the centre to the point. Thus, at 30 feet from the centre, or 20 feet from the abutment, the compression in the case assumed above becomes —

$$S = \sqrt{\left(\frac{1 \times 100 \times 100}{8 \times 10}\right)^2 + (1 \times 30)^2} = 128.55 \text{ tons.}$$

If the roadway was simply suspended by verticals from the arch, the tension on the lower chord would be uniform throughout. The introduction of the diagonal bracing, however, required by the moving load, causes the tension to decrease slightly towards the abutments. The maximum strain, however, upon the chords, will occur when the bridge is uniformly loaded, and this strain is given by the formula above. We have thus only to find the compression at the centre of the arch, and take the same as the tension throughout the lower chord, and for other parts of the arch use the second rule above given.

The strain upon the bracing of the Bowstring Girder is more difficult to obtain, on account of the continually changing angle of the braces themselves, and also of the upper chord or arch. It is much less than the strain upon the braces of a truss with parallel chords, and increases from the ends to the centre, instead of from the centre to the ends. If the rolling load is equal to, or larger than the permanent weight, counterbracing will be needed for a large part of the truss. The strains upon the bracing of the various forms of the Bowstring Girder are best found by

means of a graphic resolution of forces made upon a sketch of the girder drawn to a large scale. This process consists in placing the maximum load at each apex of the braces at the lower chord, and in following the separate effect of each partial load throughout the system to each abutment. The algebraic sum of the various strains upon each brace will show the maximum requirement to be provided for, care being taken to distinguish the tensile from the compressive strains.

As before remarked, the Bowstring Girder requires more material in the chords, but less in the braces, than the ordinary truss. Upon the whole, the difference is rather in favor of the truss with parallel chords; and, as the Bowstring is decidedly inferior in facility for lateral bracing, and for anchoring to the masonry, it has not been thought advisable to consume space here to illustrate the method, necessarily somewhat tedious, of obtaining the strains upon the web.*

An examination of the Bowstring Girder suggests a modification of the truss with parallel chords; viz., the arching of the top chord. By so doing we increase to a slight extent the compression at the ends of the upper chord, and at the same time reduce the thrust upon the braces towards the end. The extra strain thus put upon the ends of the chord does not require an increase of material in that member, as the practical requirements make

* The reader is referred to Mr. Stoney's work, Vol. I. pp. 131–138, for remarks upon the Bowstring Girder and the Braced Arch. Also to Unwin's Iron Bridges and Roofs, pp. 76–80, and pp. 165–176. Particular reference to obtaining the strains upon the Bowstring Girder will also be found in an exceedingly plain essay upon The Strains on Structures of Ironwork, by F. W. Shields, London, 1867. Prof. Rankine, Civ. Eng. pp. 482, 483, and 562–570, refers to the same subject. Mr. Whipple, pp. 16–20, compares the amount of material in the Bowstring and in the Truss with parallel chords. Illustrations of the Windsor and Shannon Bridges will be found in Mr. Humber's large work on Bridges. An excellent example of the application of the graphic method of obtaining the strains upon a braced arch, is given in a valuable paper in the Franklin Journal for 1870, by Mr. Jos. M. Wilson, describing the wrought iron elliptical arch upon the Pennsylvania Railroad over Thirtieth Street, Philadelphia.

it more than sufficient for the simple thrust when the chords are parallel. The reduction of the length of the braces towards the end, however, enables us to reduce their sectional area, while the maximum strain upon the chords is governed by the height at the centre. The curved upper chord, moreover, presents a pleasing appearance to the eye. The calculations for this form of truss may be the same as if the chords were parallel, though the result will err somewhat, but on the side of safety. The larger spans of the fine bridge recently completed across the Missouri River at Kansas City, and referred to in a following chapter, are built upon this plan.

It was at one time thought advantageous to connect the adjacent spans of long bridges. This operation is now very generally abandoned, the slight advantage gained being more than balanced by the reversion of strains thus introduced. If two adjacent spans are made in one piece, and then placed upon two abutments and an intermediate pier, the strains at the centre of the spans will be reversed over the pier, the top chord at that point suffering extension, and the lower one compression, while at a certain distance from the pier there will be no strain at all upon the chords. This point of no strain is not fixed, but depends upon the amount and position of the passing load. If the weight was all permanent, or if the permanent load was very large compared with the passing weight, the saving by connecting the adjoining spans would be considerable, as the chord strains would be materially reduced; but in ordinary spans the permanent weight is not large enough in comparison with the moving load to render the advantage of connecting the spans sufficient for its adoption. The effect of a heavy load passing over one of several continuous spans is to depress the span it is on, and to elevate the adjoining ones, thus reversing the strains upon the latter, and the several parts of the frame must be able to bear this reversion.*

* Mr. Stoney, Vol. I. p. 17. states that an extraordinary load upon the centre span of the Boyne Viaduct (a continuous girder over the River Boyne, on

Compound Trusses.

In the preceding figures, 77, 79, and 83, we have shown several plans in common use, in which a combination is made simply by the multiplication of single systems. The strains in the chords are, of course, the same, whether the system be single or double, or closer, though the decrease of strain from the centre towards the end is less abrupt in a compound system; but the strains upon the braces will be approximately half as much in a double as in a single system, a third as much in a triple system, and so on, so that we have only to estimate the strains for a single system, and then divide them by the number, to obtain the strains upon the members of the compound one. This proceeding gives results very nearly correct. The actual examples of compound trusses given in advance will show the method of proceeding when greater accuracy is required.

the Dublin and Belfast Junction Railway, near Drogheda, of which the central span is 264′ and the end spans 138′ 8″) raised the ends of the side spans off from the abutments. A load sufficient to depress one of the tubes of the Victoria Bridge (the spans of which are from 242 to 247 feet) $\frac{3}{8}$ of an inch, raised the adjoining tube $\frac{3}{8}$ of an inch. (Hodge, Victoria Bridge, London, 1860.) Mr. Clark states that a load in one of the large spans of the Britannia Bridge (the large spans being 460 feet nearly, and the small ones 230 feet) sufficient to cause a depression of 0.676 inches, raised the centre of the adjoining large tube 0.19 inch, and that of the neighboring small tube 0.109 inch. (Britannia and Conway Tubular Bridges, Vol II. pp. 704–706.) Great care was taken in erecting the Britannia Bridge to make the work continuous. Inasmuch as the spans were raised in one piece, and thus had already assumed their permanent deflection when put upon the piers, a connection then made would have been of no use. To effect the connection, so as to secure the advantage of continuity, one end of the tube was raised, and the connection then made at the other, when by letting the first end down to its position an initial strain was thrown upon the connection, thus, according to Mr. Clark, bringing every part of the tube into the same strain as if it had been framed in one piece. For discussions upon this matter, see Rankine, Civ. Eng., pp. 287–292; Stoney, Vol. I. chap. X., and the latter part of Vol. II. of Mr. Clark's great work upon the Britannia and Conway Bridges.

Boiler Plate Girders.

From the double or triple system, we pass easily to the extreme form of the compound girder, viz., that in which the web is continuous, and made of boiler plate. In this form of bridge the chord strains may be obtained by the rules already given. The strains upon the web, however, are somewhat indefinite. No matter how thin the web is made, it will probably be strong enough for the tensile strains coming upon it; but the single plate web alone is rarely sufficient for the compressive strains. It requires in all cases to be stiffened by T, or angle irons, running vertically on both sides of it, from the top to the bottom flange. These ribs may be regarded as the compression members of the web, and may be at the same time used as the splices for the vertical joints of the plates. Practical considerations prohibit the use of plates less than $\frac{1}{4}$ of an inch in thickness; these plates should, of course, increase in thickness from the centre to the ends, and may be stated generally to be as follows: —

Span 50 feet, centre plates $\frac{1}{4}$; end plates $\frac{3}{8}$.
Span 75 feet, centre plates $\frac{3}{8}$; end plates $\frac{1}{2}$.
Span 100 feet, centre plates $\frac{1}{2}$; end plates $\frac{5}{8}$.

The amount of angle iron to be added will also increase from the centre to the ends. The dimensions of boiler plate bridges, from actual practice, will be seen in a following chapter.

The advantages of the boiler plate girder are confined entirely to small and shallow bridges. Above 60 feet span, or thereabouts, it is inferior in point of economy to the open truss: and the larger the span the greater its disadvantage. Notwithstanding the appearance of rigidity presented by the boiler plate web, it is found in practice to be in no way superior to the open bracing.*

* Stoney, Vol. I. pp. 142, 143; Vol. II. p. 321. Unwin, p. 68.

CHAPTER XI.

BRIDGE-BUILDING.

Illustrations from Practice.

The preceding observations upon the strength of materials and upon the strains in girders, may now be applied to the designing of bridges.

It will be necessary in the first place to consider the amount and character of the load to which railway bridges are subjected. An ordinary locomotive weighs about $1\frac{1}{4}$ tons per foot of its length; though there are many freight engines in this country of double that weight. A train of cars will rarely weigh more than $\frac{1}{2}$ a ton per foot, except in the case of mineral trains, which may reach $\frac{3}{4}$ of a ton. To the rolling load is to be added the dead weight, or the weight of the bridge itself, which of course depends upon its length. Col. 4, of the table following, is prepared from the actual weights of a large number of railway bridges, and shows the weight of the structure itself per foot, increasing, of course, with the span. The weight per foot, however, of the rolling load grows less as the span increases. Col. 2, in the table, shows the maximum load which ordinary practice will put upon the several spans. A first-class passenger engine will have 10 tons upon each pair of drivers, and this 10 tons will be applied at a single point. If the span is 5 feet, as in the case of a cattle-guard, this will be 2 tons per running foot, applied at the centre, which is equivalent to 4 tons per foot distributed over the length of the girder. In the 10 feet span, 20 tons may be put upon a length of 6 feet, by two pairs of drivers, which may be called equivalent to a distributed load of 3 tons per foot. The eight-wheeled connected

freight engine, built by the Baldwin Locomotive Works, termed the "Consolidation" pattern, has from 75,000 to 80,000 pounds on a length of $14\frac{1}{2}$ feet of wheel base, and this load would cover only half of a 30-feet girder. The severest loads to which small bridges can be subjected will be when one of these heavy engines is on the middle of it. Upon a 100-feet bridge we may occasionally have two heavy engines with their tenders, weighing in all 120 tons. It is rare that more than two engines will be attached to one train; in which case the maximum load upon any bridge over 100 feet span may be taken at 120 tons for 100 feet, and $\frac{3}{4}$ of a ton per foot for the remainder.

In this manner the loads for the spans above 100 feet in the table are obtained. It has already been remarked that a rolling load may be regarded as equivalent to a dead load of double the amount; but this increased effect of a moving weight applies only to small bridges. Above about 80 feet it may be disregarded.

In the table, the numbers in col. 5 for 100 feet span and upwards are obtained by simply adding col. 3 to the average of the numbers in col. 4. For the spans below 100 feet a certain percentage has been added to the live load to reduce it to an equivalent dead load; and finally, the average of the numbers in col. 4 added as before. These percentages are 0.1 for 75 feet, 0.2 for 50, 0.3 for 40, and so on to 0.8 for a span of 5 feet.

Table of Loads for Railway Bridges.

Span in Feet.	Total Live Load applied in Tons.	Equivalent Distributed Load per Foot in Tons.	Dead Load per Foot in Tons.	Total Weight in Tons per Foot.
5	10	4.00	0.1 to 0.2	7.35
10	20	3.00	0.1 " 0.2	5.25
15	30	2.00	0.1 " 0.2	3.35
20	35	1.75	0.2 " 0.3	2.87
30	45	1.50	0.2 " 0.3	2.35
40	55	1.37	0.3 " 0.4	2.13

Span in Feet.	Total Live Load applied in Tons.	Equivalent Distributed Load per Foot in Tons.	Dead Load per Foot in Tons.	Total Weight in Tons per Foot.
50	65	1.30	0.3 to 0.4	1.91
75	93	1.24	0.4 " 0.5	1.81
100	120	1.20	0.5 " 0.6	1.75
150	157	1.05	0.6 " 0.7	1.70
200	195	0.97	0.7 " 0.8	1.72
250	232	0.93	0.8 " 1.0	1.83
300	270	0.90	1.0 " 1.2	2.00

Short girders lack the weight to prevent springing under a rapidly passing and vibrating load, and they should be well stiffened, both laterally and vertically, and tied to heavy and well-bedded wall plates, both for their own sake and also to protect the masonry. In bridges of any considerable size the greatest strain upon the chords, and the braces or ties towards the ends, will arise from a uniformly distributed load; but the maximum strain upon the ties and braces towards the centre, and upon the counterbraces and the floor beams, will arise from the concentrated load of a heavy locomotive, or a pair of locomotives. It is necessary in proportioning bridges to regard carefully the action of this concentrated load as it passes.

Bridges for Small Spans.

Suppose now that we would span openings from 5 to 20 feet, with a single stick of white pine under each rail. The rule in Chap. VIII. gives, in connection with the weights given in the table on page 213, the following, in which the breadth of the stick is in each case 12 inches.

$$\text{Span 5 feet: } d = \sqrt{\frac{5 \times 20{,}580 \times 5 \times 12}{4 \times 12 \times 1200}} = 10\tfrac{1}{3} \text{ inches.}$$

Span 10 feet: $d = \sqrt{\dfrac{5 \times 29{,}400 \times 10 \times 12}{4 \times 12 \times 1200}} = 17\tfrac{1}{2}$ inches.

Span 15 feet: $d = \sqrt{\dfrac{5 \times 28{,}140 \times 15 \times 12}{4 \times 12 \times 1200}} = 21$ inches.

Span 20 feet: $d = \sqrt{\dfrac{5 \times 32{,}144 \times 20 \times 12}{4 \times 12 \times 1200}} = 25\tfrac{7}{8}$ inches.

The value of W in the above is obtained by taking half the product of cols. 1 and 5, in the preceding table, as the load applied at the centre, and half of that again for each side timber, and multiplying it by 5 as the factor of safety. It will be seen by the results that it would not be feasible to exceed about 15 feet with a single unsupported beam. The large amount of material in a timber 20 or 25 inches deep can be better employed when cut up into smaller sizes and used as braces. The method of arranging the small spans above will be seen by reference to Plate V.

Trussed Girders.

When the opening exceeds 15 or 20 feet, the girder should be trussed by rods beneath, or supported by braces above. The principles of calculation for the several small spans shown in Plate V., are the same as for the larger spans referred to in a former chapter, care being taken to assume a high working load, as in the table on page 213. Especial care, as before remarked, must be taken to make all of the connections of these small bridges strong and firm, and to anchor them well to the masonry; they should also be made very stiff laterally by sufficient bracing. The rods of the simple trussed girders, Figs. 11 and 12, are to be regarded as the tension members of the system, and treated as the end panel rods in a Pratt Bridge. This will give them a much larger area than is generally seen in practice, but the rods in the majority of these girders in actual use are strained beyond the safe limit by every train that passes.

Let the span, Fig. 11, Plate V., be 20 feet, and the depth, or the vertical reach of the truss rod, be 20 inches. The weight per foot, in col. 5 of the table, is 2.87 tons, or 57.40 tons of equivalent distributed load in all, or 28.70 on each truss; equivalent to 14.35 applied at the centre, or 7.17 tons upon each half of the rod; this multiplied by the angle of the rod, or by $\frac{122}{20}$, gives 43.73 tons as the direct strain on the rods, which at 5 tons per square inch, requires 9 inches of area very nearly; and if the rods or bars are in pairs on each side of the piece to be trussed, $4\frac{1}{2}$ inches area in each, which is obtained by a rod $2\frac{3}{8}$ inches in diameter. This is a much larger rod than is commonly found in practice, but none too large for the requirement.

If in Fig. 12 we should take the maximum load that can be brought over each of the intermediate points of support as 20 tons, or 10 tons on each truss, and if we call the horizontal reach 7 feet, and the vertical reach as before 20 inches, we have $10 \times \frac{86}{20}$ or 43 tons for the strain upon each set of truss rods. These examples show a lack of depth in the trussing, and suggest an arrangement like Fig. 13. Calling the horizontal reach of the rods here 13 feet, the span being 25 feet and the vertical reach 5 feet, and the equivalent distributed load 2.5 tons per foot, we have $2\frac{1}{2} \times 25 = 62.50$, or upon each truss 31.25; the equivalent load at centre is 15.62, or on each half 7.81 tons; and finally $7.81 \times \frac{14}{5} = 21.9$ tons of direct strain, requiring 4 inches of sectional area, which is obtained by two rods of $1\frac{5}{8}$ inches diameter.

In Fig. 14, the span and depth being the same as above, the strain upon the rods will be less on account of their greater inclination. The strain upon the horizontal tension bar will be as shown in the resolution of forces in Fig. 70. A truss like Fig. 14 requires the diagonals A D and C B for counterbracing. A load placed at A would cause the point C to rise, thus shortening the diagonal A D and lengthening C B; a movement which may be checked by a tie B C or a brace A D. With the load at C, the strains, of course, are reversed. A pair of ties, therefore, as in the

drawing will be needed. The tie B C should be able to transfer the $\frac{1}{3}$ of the load at A to the farther abutment. In Fig. 15 there is no need of the counterbracing; but the load at each post is divided unequally. If we have 10 tons at B, $\frac{2}{3}$ of it go to A, and $\frac{1}{3}$ of it to C; the $\frac{2}{3}$ require to be increased for the inclination of A B, and the $\frac{1}{3}$ for the inclination of B C. Fig. 16 is simply a Fink truss, already illustrated. Fig. 17 is to be treated precisely in the same manner as Fig. 14, $\frac{3}{4}$ of the load at B going to the near abutment, and $\frac{1}{4}$ to the distant one, the $\frac{1}{4}$ being taken by the counter rod B C. The strains upon the other parts of this arrangement are to be found as already shown for the Whipple Bridge in Chapter X. The strains upon the braced girders, Figs. 1 to 10, are to be obtained by the rules already given. The horizontal timber in these several forms must be large enough to have the proper stiffness between the intermediate points of support. A stiff track stringer, like that shown in Fig. 9, will do good service in distributing a heavy concentrated load over the girder, and thus protect the principal stringers from a severe cross strain. These small bridges should be bolted to the wall plates, and the latter fastened to the masonry. In proportioning the parts of the joints, the remarks in Chapter IX. should be borne in mind.

Figs. 18 to 24 show the details of a trussed rolled beam, seen in section at Fig. 18. Fig 19 is the connection between the truss rod and girder, with a filling piece, seen also in Fig. 18, to bring the rod outside of the lower flange. Fig. 20 shows the top and bottom of the centre post. Fig. 21 is a transverse section, showing lateral bracing. Figs. 22, 23, 24, the chair for the end bearing. Fig. 25 represents a small span formed of four rolled I beams, as made by the Phœnixville Bridge Company, where the depth between the grade of the road and the bottom of the bridge is very limited, the track stringers resting upon wrought iron cross beams. Figs. 26 to 38 show details for small bridges, and explain themselves.

The Howe Bridge of Wood.

To pass to larger spans, Plate VI. shows the general arrangement and the details of the standard form of the Howe Truss, with the braces and both chords of wood. This form of truss has been perhaps more widely adopted in the United States than any other, and when protected from the weather makes an economical and good bridge for ordinary spans. For large spans, i. e., 200 feet and upwards, an arch has frequently been applied to the truss with good results; though it is not easy to make different systems thus combined act in harmony. In many of the examples seen upon railways, however, the proportions are not what a correctly designed bridge should have. The section of the chords is generally the same, and the splicing the same, from the centre to the ends. As far as the simple tension or compression is concerned, this involves a waste of material at the ends, or a lack of strength at the centre. As the chords, however, support the floor beams, and as it might cost more to vary the section in such a material as wood, from point to point, according to the strain, than to employ the excess of timber at the ends, it is probable that the commonly adopted plan is best. It will be seen by the Plate, Figs. 5 and 6, that the outer spaces are 1½ inches, and the middle space ½ inch. The clamps are all placed in the outer spaces, there being one clamp and four keys in each panel. The disposition of the joints is shown in Figs. 3 and 4, the clamp being shown enlarged in Fig. 10. An essential part of this truss is the casting shown in Fig. 9, and also shown in Figs. 7 and 8. The upright tubes in Fig. 9 pass between the stringers, and form a rigid connection between the thrust of the braces and the pull of the ties, without in any way crushing the chords crosswise. The details given in the Plate should be carefully examined by the reader, as they show the Howe Truss as actually built by a firm whose practice may be supposed to show that plan in its best aspect.

The Howe Bridge of Wood and Iron.

Plate VII. represents a bridge of 300 feet span, built in 1868, under the direction of Mr. Frederick H. Smith, of the Baltimore Bridge Company, over Laughery Creek, near Aurora, Indiana. This is not a railway bridge, but it has many features applicable to railway work, and is throughout an instructive example, the span being unusually long for a wooden structure.

The general plan is the same as the Howe Truss, but the lower chord is of wrought iron, and completely adjustable, while the connections of the vertical and lateral systems with the chords, shown in Figs. 7, 8, and 10, are such that the wooden portions of the structure can be removed and replaced without the use of false work.

The ties and braces in this bridge are proportioned to carry the weight of structure plus a rolling load of 1024 pounds per lineal foot. The weight of the structure is greater than that of the load, and no counterstrains arise; but counterbraces are used to stiffen the braces. The chords are proportioned to carry the bridge when uniformly loaded, the lower one decreasing in section from the centre to the ends, while the upper chord retains the full centre size throughout. The factor of safety assumed is 4. The depth of truss is 30 feet at the ends, and 31 feet at the centre. The panels, of which there are 16, are 18 feet and 9 inches long. The width of the bridge is 23 feet. The load assumed is 1024 × 300, or 307,200 pounds. The weight of the bridge is 396,800 pounds, and the total 704,000 pounds, or on each truss 22,000 pounds per panel. With these data, the strains, as given by the builders, are as shown in the tables on page 220.

The maximum compression upon the upper chords is 433,000 pounds, which, divided by 850, gives 510 inches, obtained by 4 sticks 8 × 16 inches. The counterbraces are 6 × 10 throughout; the end posts are 8 × 8, in pairs. The lateral struts are 8 × 9; the floor girders 6 × 16, in pairs; the floor joists 12 × $2\frac{1}{2}$, and the planking 8 × 2. The lateral rods are $1\frac{1}{4}$ inches in the first four panels from the abutments, and 1 inch in the remainder.

The strains below upon the iron work are divided by 15,000 to obtain the sectional areas of the members. The strains upon the wooden parts are, as seen below, divided by factors varying from 825 pounds for the end braces to 215 pounds per inch for the middle ones, in accordance with the remarks in a preceding chapter upon factors of safety.

Chords and Ties.

Lower Chord.			Rods.		Vertical Rods.			Rods.	
Panel.	Tension.	Section.	No.	Diam.	No.	Tension.	Section.	No.	Diam.
1	103,000	7.0	12	$\frac{7}{8}$	1	165,000	11.0	4	$1\frac{7}{8}$
2	193,000	13.0	12	$1\frac{1}{4}$	2	144,000	9.6	4	$1\frac{3}{4}$
3	268,000	18.0	12	$1\frac{3}{8}$	3	123,000	8.2	4	$1\frac{5}{8}$
4	330,000	22.0	12	$1\frac{1}{2}$	4	104,000	7.0	4	$1\frac{1}{2}$
5	378,000	25.2	12	$1\frac{5}{8}$	5	83,000	5.5	4	$1\frac{3}{8}$
6	413,000	27.5	12	$1\frac{3}{4}$	6	64,000	4.3	4	$1\frac{1}{4}$
7	433,000	29.0	12	$1\frac{3}{4}$	7	46,000	3.1	4	$1\frac{1}{8}$
8	440,000	29.4	12	$1\frac{3}{4}$	8	34,000	2.3	4	1

Braces.

No.	Compression.	Factor.	Section.	No. of Pieces.	Size.
1	198,000	825	240	2	12 × 10
2	173,000	750	230	2	12 × $9\frac{1}{2}$
3	148,000	680	215	2	12 × 9
4	124,000	620	200	2	12 × $8\frac{1}{2}$
5	100,000	525	190	2	12 × 8
6	77,000	425	180	2	12 × $7\frac{1}{2}$
7	55,000	325	170	2	12 × 7
8	34,000	215	160	2	12 × $6\frac{1}{2}$

In Plate VII., Figs. 2 and 3 show the application of the end post to the abutment; Figs. 4 and 5, the connections of braces and lower chords; Figs. 6, 7, and 8, the connections with top chord; and Fig. 9, detail of lateral braces.

Quadrangular Truss by Kellogg and Maurice.

Plate VIII. shows a side elevation, and plan, and Plate IX. the details of a quadrangular truss with wooden chords. The panels in this truss are square, but the lower chord is supported at intermediate points half way between the posts. The details are especially good, and the method of reducing the sectional area of the lower chords towards the ends of the truss, in accordance with the decreasing strain, is at once effective and simple. This plan has been adopted in many places, and has given entire satisfaction.

The Pratt Bridge of Wood and Iron.

Plate X. represents one span of the Vermilion River Bridge, at Danville, Ill., upon the Toledo, Wabash, and Western Railway, built in 1868, by the Detroit Bridge and Iron Company. The span shown is 197′ 4″ from centre to centre of the end posts; the height, from centre to centre, 24 feet, the panels, 16 in number, 12′ 4″ each; and the width, from centre to centre of trusses, 14 feet. The dead load is taken at 1000 pounds per foot, and the live load at 2340, or 3340 pounds per foot in all, or 20,600 pounds per panel of each truss. With the above data, the strains upon the several parts are as figured below. Fig. 1 in the Plate, shows the general elevation of one span; Figs. 2, 3, and 4, enlarged side views of the upper chord at the end, over the first post, and over the centre post; Fig. 5 shows the arrangement of the angle block and key between the stringers; Fig 6, a plan of the same; Fig 7, side view of lower end of centre post; Fig. 8, plan of washer on upper side of top chord; Fig. 9, lower

end of first post from the abutment, and Fig. 10, seat for foot of post; Fig. 11, transverse section through top chord and foot of post at end of bridge; and Fig. 12, same at centre; Fig. 13 shows a plan of the arrangement of the chord links, lower transverse struts, and diagonal ties, at the centre of the span.

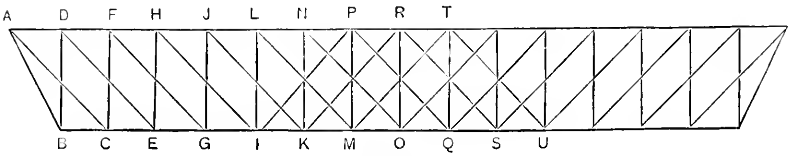


Fig. 103.

The strains upon the different parts, and the dimensions of the several members are given by the builders as follows: —

Tension Bars.

No. of Bar.	Strain.	Area.	Size.
A B	93,000	8.00	2 Bars 2 × 2
A C	103,000	9.05	2 " $2\frac{1}{8} \times 2\frac{1}{8}$
D E	89,000	8.00	2 " 2 × 2
F G	74,000	7.03	2 " $1\frac{7}{8} \times 1\frac{7}{8}$
H I	59,000	6.12	2 " $1\frac{3}{4} \times 1\frac{3}{4}$
J K	44,000	4.59	2 " $1\frac{1}{2} \times 1\frac{1}{2}$
L M	30,000	3.12	2 " $1\frac{1}{4} \times 1\frac{1}{4}$
N O	29,000	3.00	2 " $1\frac{1}{2} \times 1$
P Q	21,000	2.00	2 " 1 × 1
R S	14,000	1.50	2 " $1 \times \frac{3}{4}$
T U	12,000	1.25	2 " $1 \times \frac{5}{8}$

Posts.

No. of Post.	Compression.	Area.	Size.
D B	82,000	168	1 Post 12 × 14
F C	72,000	168	1 " 12 × 14
H E	62,000	144	1 " 12 × 12
J G	52,000	144	1 " 12 × 12
L I	41,000	100	1 " 10 × 10
N K	31,000	100	1 " 10 × 10
P M	21,000	100	1 " 10 × 10
R O	21,000	100	1 " 10 × 10

Lower Chords.

Section.	Strain.	Area.	Size.
B C	42,000	3.78	3 Bars $1\frac{3}{8}$
C E	116,000	10.56	4 " $1\frac{5}{8}$
E G	180,000	15.30	5 " $1\frac{3}{4}$
G I	233,000	18.36	6 " $1\frac{3}{4}$
I K	275,000	24.00	6 " 2
K M	307,000	25.06	6 " $2\frac{1}{8}$
M O	328,000	30.36	6 " $2\frac{1}{4}$

The chord pins are $3\frac{1}{2}$ inches in diameter. The top chord, which is of pine, has an area throughout of 288 square inches, being composed of 2 pieces 6 × 12 and 1 piece 12 × 12, the spaces being $2\frac{1}{2}$ inches. The floor beams are 6 × 13, placed 2 feet apart from centre to centre. The lateral struts are 4 × 6, and the rods $1\frac{1}{4}$ inches in diameter.

The following table gives the quantities in one span :—

Items.	Feet. Board Measure.	Wrought Iron. lbs.	Cast Iron. lbs.
Top Chords, ...	10,000	790	3,049
Posts, ...	7,200	480	22,950
Lower Chords, ...	. . .	24,932	. . .
Main Ties, ...	. . .	22,516	2,410
Counter Ties, ...	. . .	1,872	. . .
Lateral System, ...	1,800	6,702	510
Coupling Pins, ...	. . .	3,100	156
Floor Beams, ...	10,400	670	268
Track Stringers, ...	4,800	. . .	. . .
Totals, ...	34,200	61,062	29,343

The above work is unsurpassed as a combined structure of iron and wood. It removes the prominent defects of the old-fashioned Pratt Truss, viz., the wooden lower chord, and the crushing of the top chord between the washer and the post.

The Kansas City Bridge.

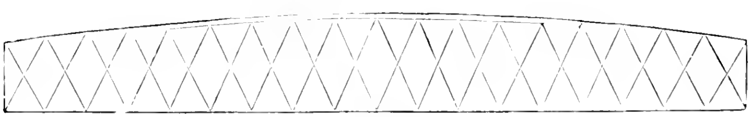


Fig. 104.

Fig. 104 shows the general plan of the longest span of the Kansas City Bridge. It is 246 feet between the centres of the end connections, 22 feet high from centre to centre at the ends, and 31.25 feet high at the centre. By thus curving the top chord a great depth of truss is obtained at the centre without increasing

the length of the heavy ties and braces at the ends. The general plan, as seen by the plate, is a double triangular or trellis girder, the top chord and braces being of wood, the lower chord and ties of wrought iron. The braces are in pairs, the counterbraces single, passing between them. In the panels near the ends there are four main ties, two passing outside of the main braces, and two between them and the counterbrace. In the centre panels the main ties are in pairs, passing outside of the main braces, while the counter ties pass between the main and counterbraces. The counterbraces are carried throughout the truss, to take a bearing in screwing up the main ties; but the counter ties are carried only so far as the stiffness requires. The upper chord is made of five pieces, with a sub-chord at the central part made of two pieces. All of the work is so arranged that the several parts may be renewed or replaced without the use of false work. The lower chords are the Linville and Piper patent upset eye bars, and the ties, of square iron, are made with a welded loop at the lower end to pass round the chord pin, while the upper end is upset and enlarged for a screw. The end posts and braces bear on cast iron pedestals resting on a wall plate, also of cast iron, carefully fitted to the masonry, and bedded in mortar. At one end rollers are placed between the pedestal and the plate. The ends of the braces are cut with two faces, making an obtuse angle with each other, and the angle blocks being cast to correspond, the brace can never slip upon its bearing. The connection between the braces, ties, and top chord is of cast iron, and consists of three members, as shown in Fig. 105. First, the angle block proper, A, which is placed below the chord, and receives the end of the braces. Second, the keys, B, which pass through the chord in the same manner as the ordinary packing blocks; these are cast hollow, in as many parts as there are spaces between the stringers, and with side plates to receive the ends of the timbers whenever a joint is broken. Third, the washer plates, C, which rest on the top of the chord, and carry the nuts of the ties, the plates for the main and counter ties being cast separately. The

angle blocks are cast with a flange running their whole length, which fits into grooves cut across the under side of the chords, and bears against the keys. The ties pass through the hollow keys, nowhere coming in contact with the wood. By the above arrangement the strains are carried without in any way crushing

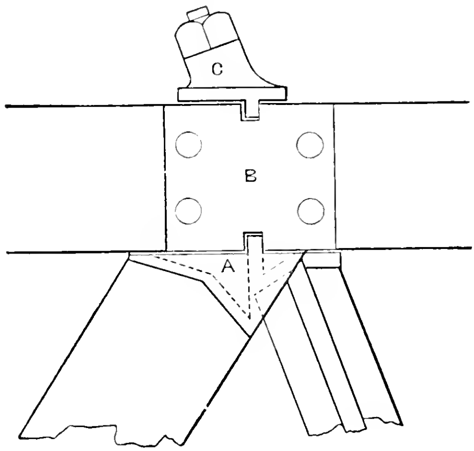


Fig. 105.

the timber of the chord across the fibre. The lower angle block is cast in one piece, having a series of webs on the under side through which the pin passes. The top laterals are similar to those used in the Howe Truss, except that the bearing of the half struts is taken by small castings placed around the centre of the long strut, instead of being thrown directly upon the wood. The bottom laterals have cross struts and diagonal ties, each strut extending from the foot of a post to the opposite point on the chord link of the other truss (the bridge being oblique). The ties connect at one side with an eye plate, which fits over the chord pin, and at the other with bent rods attached by nuts and a casting to the chord link, each tie being in two parts, the adjustment being made by a sleeve nut. The camber of the above span was $8\frac{1}{2}$ inches; the extreme vertical range from the lowest temperature, and no load, to the highest temperature, and a maximum

load being $4\frac{15}{32}$ inches. A rolling load of 2240 pounds per foot, or 250 tons for the whole span from centre to centre of piers, produces a compression on the top chord of 325 pounds per inch of section, and a tension on the lower chord of 5200 pounds per inch. The compression upon the braces is 240 pounds per inch, and the tension on the ties 5000 pounds; these strains being in addition to those produced by the permanent or dead load. In proportioning the bridge, the central tie rods and the truss rods of the floor beams were allowed to bear 10,000 pounds per inch, the end ties and chord links 12,000, the top chords 800 pounds, and a factor of safety of 7 was assumed for the braces. The trusses are anchored to the piers by one and a half inch round irons, extending from the top chords and passing over cast iron struts projecting out from the coping, and fastened by a nut and screw through the eye of a 3 inch pin, let 15 inches into the masonry. The amount of material in the 248 feet span, including floor beams and stringers, was 101,688 feet B. M. of timber, 147,432 pounds of wrought, and 70,646 pounds of cast iron. The strains upon the several members from a dead load of 1350 pounds, and a live load of 1120 pounds per foot on each truss are as below.

No. of Panel.	Upper Chord.	Lower Chord	Main Brace.	Main Tie.	Counter-braces.	Counter Ties.
1	86,600	71,514	164,400	172,200	. . .	. . .
2	225,750	201,363	146,700	153,700	. . .	. . .
3	350,600	304,993	129,200	142,400	. . .	. . .
4	417,500	389,445	118,075	123,800	. . .	. . .
5	482,575	454,115	98,120	113,450	. . .	. . .
6	530,000	506,385	86,110	95,200	. . .	24,860
7	568,000	565,400	65,800	84,410	23,900	24,900
8	588,050	572,560	51,100	65,650	24,650	33,400
9	593,000	583,973	34,300	50,860	33,320	34,300

The preceding is an excellent example of a composite truss of wood and iron, the whole structure, both in its general proportions and in its detail, having been designed with great care.*

MR. POST'S BRIDGE.

Plate XI. represents the general plan of bridge designed by S. S. Post, Esq. In Fig. 1 the top chord and posts are of wood, the lower chord and ties of iron. In this bridge the posts are inclined at what the inventor considers the most economical angle. By this inclination the ties have a run of two panels, and the counter ties of one, though from the position of the posts the inclination of the two sets of rods is the same. Fig. 2 shows a plan of the lower chord, floor, and lateral bracing, at the end of the bridge. Fig. 3 is a part elevation of the same truss, entirely of iron; the section of the end post being shown in Fig. 4, and the section of the brace in Fig. 5. Figs. 6 and 7 show the connection at the lower end of the post. Figs. 8, 9, and 10 are different views of the floor suspenders. Fig. 11 shows the end of a pair of floor girders, and the angle block for the lateral ties, the same being shown in elevation in Fig. 12. Fig. 13 shows the cutting out of the upper flange of the floor girder for the passage of the suspension link; this link being made of one inch square iron for railway bridges, with panels from 11 to 13 feet in length. A bridge upon this plan, made of iron, upon the Chicago and Alton Railroad, over the Kankakee River, at Wilmington, the spans being 104 feet, was first loaded with 2270 pounds per foot, when the counter rods were brought to a bearing, in which condition two locomotives, weighing with their tenders 50 tons each, produced a depression of from $\frac{1}{4}$ to $\frac{5}{16}$ of an inch. The same engines coupled together, and run across the bridge at

* The reader is especially referred to the very elaborate description of the above bridge by its engineers, Messrs. Chanute and Morrison. It was intended to devote a plate in this volume to the larger span; but the ample illustrations in the work of its builders render such a proceeding needless.

different speeds, in some cases as high as 25 miles an hour, produced a depression of $\frac{1}{8}$ inch. A span loaded with 100 tons of rails, besides the locomotives, was depressed $\frac{7}{16}$ of an inch.

MR. FINK'S BRIDGES.

Plate XII. gives, from Figs. 1 to 7, sketches of the iron triangular bridge, built by Mr. Fink, across the Cumberland River, at Nashville, Tenn., upon the Louisville and Nashville Railroad. It consists of two spans of 210 feet in length, and a pivot draw 280 feet long in all. Fig. 2 is an enlarged transverse section of a part of the floor, showing the trussing of the floor beam; Fig. 3, a plan of the end of the floor beam, showing connection of lateral ties and thickening plates for the pin at the upper end of the truss rod. Figs. 4 to 7 explain themselves.

Fig. 8 is a portion of the triangular bridge of iron and wood combined, built also by Mr. Fink, over the Tennessee River, on the Memphis and Charleston Railway, at Decatur, Ala. It consists of 10 spans, each 155 feet, and a double draw 140 feet long in all. The several sketches represented in the plate, in Fig. 13, refer to the corresponding letters on Fig. 8. In the same figure, A shows the seat upon the masonry, L, M and N top chord plans at B, E and G, and C the lower end of the vertical floor suspender. The enlarged details from Fig. 9 to Fig. 12 explain themselves.

An improved form of a combined wooden and iron triangular truss, by Mr. Fink, is shown in Plate XIII. The reader's attention is called to the manner in which the counterstrains are provided for. The weight of the truss is sufficient to neutralize the counterstrains arising from a partial load beyond the centre braces H G, and the adjoining ties H E; and only in these members is it necessary to provide for both strains. The figures will show the manner in which this is done. It will be seen that all parts of the truss subject to tension are of iron. By properly protecting the wood-work this plan is little inferior to a bridge entirely

of iron, while in point of cost it is not much more expensive than a wooden one. The design allows of removing any piece of timber without the use of scaffolding. This plan is being rapidly adopted in the Southern States, where several fine examples may be seen.

The Fink Bridge, which has already been referred to in Chapter X., is illustrated at length in Plates XIV., XV., and XVI. It has been adopted largely, especially in the South and Southwest, the most noted example being the great bridge across the Ohio at Louisville. This plan was first introduced about twenty years ago by Mr. Latrobe, upon the extension of the Baltimore and Ohio Railroad west of Cumberland. The longest iron bridge on that road, being across the Monongahela at Fairmount, is upon Mr. Fink's plan, and consists of three spans of 205 feet each. It was built in 1851–52, and at that time was the longest iron railroad bridge in the country.

Fig. 1, Plate XIV., shows the general elevation of the Green River Bridge, on the Louisville and Nashville Railroad. Fig. 2 is a side view of a half of one span, the dimensions of the several members being figured on the engraving. Fig. 3 is a transverse section in front of the post D, A A being the chords, P the tie for the cast iron floor beam G, X X supporting hooks for the bars of system 2 where they pass the post, Y Y brackets for supporting the bars of system 1, H strut at foot of posts, D cross rods, and N track stringers. Figs. 4, 5, and 6 are different views of the end of the chord. The span is supported on a rocker, L, to allow for variation in length arising from changes in temperature and from loads. The bearing plate K rests directly on the masonry. Fig. 7 shows the connection at the top of the middle post, G being the cast iron floor beam, and C the post proper, also shown in the section Fig. 3, the connection between the two being by means of a tenon and socket and the bolts shown in the figures. The connection between the floor beam and chord is also made by means of a tenon and socket and two bolts. Fig. 8 is a side view of the foot of the post C, and Fig. 9 a half

section and half outside view of the same; I I are cast iron keys for adjustments of the camber, and F F F connecting plates for the bars of system 1. Fig. 10 shows the detail of the end of the floor beam, and connection of the same with the chord; E is the lug for the lateral brace rods, G the floor beam forming the strut for this system, P the tie for the floor beam. Fig. 11 shows the section of the chord, and Fig. 12 a section of floor girder.

Fig. 1, Plate XV., is a side view of one span of the Barren River Bridge upon the Louisville and Nashville Railroad. Fig. 2 shows the end of chord, with rocker and bearing plate. Fig. 3 is a side view, and Fig. 4 a half plan and half section through the pin over the post D, system 1. Fig. 5 is a side view and half vertical section of chord and post B, system 2. Fig. 6 is a half bottom view and half section through pin, over post B. Figs. 7 and 8 show the connection of the top strut with the chord. Figs. 9 and 10 represent the foot of post E, and the manner of suspending the floor beam F, by a suspension link T, so that the foot of the post is free to move under the influence of load or temperature. Fig. 11 shows the mode of connecting the bars of system 1, i. e., the chain joint.

Fig. 12 is a side view, and Fig. 13 a plan of a 50 feet deck, or undergrade bridge, upon the Baltimore and Ohio Railroad. Fig. 14 is an enlarged transverse section of the same.

Plate XVI. shows several combinations of wood and iron. Fig. 1 is a side view of a through or overgrade span of 200 feet, being a portion of the Baltimore and Ohio Railway Bridge over Tygart's Valley River, at Grafton, Va., the top chords being of wood. The strut G, in connection with the diagonal rods and the floor timbers, forms the bottom system of lateral bracing. Fig. 2 shows the transverse section, L being a roof for the protection of the chord, H the strut for the top lateral bracing, and E a trussed floor beam. Fig. 3 is a side view of the chord and top of post D, and Fig. 4 a side view of the foot of the same post, E E being the floor beams, M M the bottom diagonal rods,

and R the washer on the end of the floor beams for the same. G G are longitudinal struts, and S the suspension casting. Fig. 5 shows a section of the top of post B, with the chord connection at that point. Fig. 6 is a side view of foot of post B, W being the connecting plate for the chains I I, and V cast iron adjusting keys. Fig. 7 is a side view of one of the spans of the Big Black River Bridge, on the Southern and Mississippi Railroad. In this bridge the cross ties rest directly upon the chords. Fig. 8 shows the transverse section of the same bridge. Fig. 9 is a side view, Fig. 10 a plan, and Fig. 11 an enlarged section of a 75 feet span, with wooden chords, upon the Baltimore and Ohio Railroad.

In the several examples above noticed the posts are of cast iron. In many of the more recent bridges wrought iron Phœnix columns have been used for the posts, and in some cases for the chords. Wrought iron floor beams are also employed; and in some structures wrought iron beams have been suspended, longitudinally, from post to post, with floor beams extending across the bridge resting thereon.

The largest work yet constructed upon the above plan is the fine bridge recently completed by Mr. Fink across the Ohio River at Louisville; although the channel spans of that structure are of a different form, as shown in advance.

The strains upon the members of the above bridge for a 242 feet span are determined as below, the depth of truss being 30

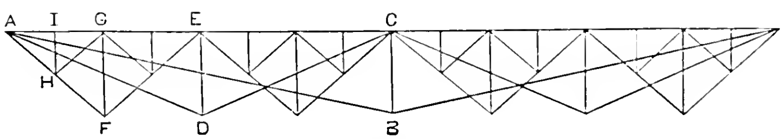


Fig. 106.

feet, the length of panel $15' \; 1\frac{1}{2}''$, and the arrangement as given in Fig. 106. The assumed moving load is 2600 pounds per lineal foot. The permanent weight from the actual bills of material employed is as follows: —

Track and flooring, 242 feet, at 690 pounds per foot, ...	166,980 pounds.
Cast iron, ...	212,528 "
Wrought iron proper, ...	179,600 "
Wrought iron columns, ...	37,180 "
Total weight of one span, ...	596,288 "
Weight of each truss, ...	298,144 "
Weight per foot of truss, ...	1,232 "
Rolling load per foot of truss, ...	1,300 "
Bridge and load per foot, ...	2,532 "

The weight supported at the feet of the several posts is as follows: —

C B (System 1), . . $\frac{242}{2} \times 2532 = 306{,}372$ pounds.
E D (System 2), . . $\frac{121}{2} \times 2532 = 153{,}186$ "
G F (System 3), . . $\frac{60.5}{2} \times 2532 = 76{,}593$ "
I H (System 4), . . $\frac{30.25}{2} \times 2532 = 38{,}296$ "

To the regular panel weight on the last system, viz., I H, an addition is to be made on account of the concentration of weight from the drivers of the engine. This addition for a panel of 15′ 1½″ has been estimated at 1704 pounds, making the total weight on the post I H 40.000 pounds. With these weights the tension upon the suspension bars of the several systems will be as follows: —

System 1, ... $153{,}186 \times \frac{124.7}{30} = 636{,}743$ pounds.
System 2, ... $76{,}593 \times \frac{67.5}{30} = 172{,}334$ "
System 3, ... $38{,}296 \times \frac{42.6}{30} = 54{,}380$ "
System 4, ... $20{,}000 \times \frac{21.3}{15} = 28{,}400$ "

The compressive strains thrown by the several systems upon the top chord are as follows: —

System 1, ...	$153{,}186 \times \frac{121}{30}$	$=$	617,850
System 2, ...	$76{,}593 \times \frac{60.5}{30}$	$=$	154,462
System 3, ...	$38{,}296 \times \frac{30.25}{30}$	$=$	38,615
System 4, ...	$20{,}000 \times \frac{15.12}{15}$	$=$	20,160
Maximum compression in chord, ...			831,087 pounds.

The chord in question is a cast iron octagonal tube, 16 inches external diameter, with a sectional area of 73.9 square inches, the factor of safety according to Hodgkinson's rule being 6.7. The posts are wrought iron columns, of the Phœnix pattern, before described, varying in section from 8.7 to 45 square inches, with factors of safety of 5 and 6.

The tension bars are of the best bridge iron, with an ultimate strength of 65,000 pounds per inch. The bars of system 1 are subjected to a maximum strain of 12,000 pounds per inch, and those of system 4 to a maximum of 9500 pounds, the strains upon the several parts being varied according to the frequency and violence of action to which they are subjected.

The preceding calculations apply particularly to the deck, or undergrade truss. In the case of the through or overgrade truss the strains in the chord and tension bars are the same, while those in each post are reduced by the weight of the moving load on one panel.

It will be seen by inspection of Fig. 106 that the Fink Truss requires no counterbracing; i. e., no introduction of special members to distribute a partial load, since it is impossible to place a load at any point on the truss where the main supporting system will not distribute it to the two abutments. Thus, if a weight W is placed at I, Fig. 106, one half of it goes to A, and one half to G; of the half carried to G, one half, or $\frac{1}{4}$ W, goes to A, and $\frac{1}{4}$ W to E; of the fourth carried to E, one half, or $\frac{1}{8}$ W, goes to A, and $\frac{1}{8}$ to C; and finally, of the weight carried to C, one half goes to A, and one-half, or $\frac{1}{16}$ W, to the farther abutment. That is, of a load placed at I, distant $\frac{1}{16}$ of the span from A, $\frac{15}{16}$ go to A, and $\frac{1}{16}$ to the unloaded abutment, which is in accordance with the law

that the weights borne by the supports are inversely as their distances from the load.

The manner in which the parts of the Fink Truss are combined, secure for it a remarkable freedom of action under the strain of moving loads or changes of temperature. The effect of a deflection produced either by a load or change of temperature is seen by Fig. 107. The feet of the posts being free to move,

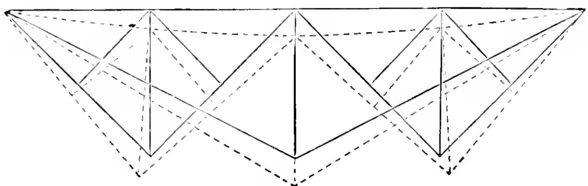


Fig. 107.

maintain their position normal to the curve of the chord, and the distance from the foot of the post to the connection of the tension bars and the chord remaining relatively the same, none of the parts are unequally strained, and the truss remains in as good adjustment as before the deflection took place. The same freedom of action exists equally in an overgrade bridge; for, although the floor is suspended from the bottom of the posts, the means of suspension are such as to allow the posts their proper motion. The self-adjustability of this truss allows the use of wood, cast and wrought iron, in combination with a certainty of correct action in all cases; for whatever be the ratio of the compression of the chord and post to the elongation of the tension bars, or the varied effect of heat on the different parts, the relative lengths of similar parts are preserved, and the truss remains in complete adjustment.

The Louisville Bridge.

This great work across the Ohio River was designed and built by Albert Fink, chief engineer, and F. W. Vaughan, principal assistant.

It consists of 29 spans, as below, commencing at the Kentucky shore, the lengths given being from centre to centre of piers: —

1 Span in Kentucky abutment,	32.5 feet.
2 Spans of 50 feet,	100.0 "
1 Pivot Draw over Canal (2 spans),	264.0 "
4 Spans of 149.6 feet,	598.4 "
2 Spans of 180 feet,	360.0 "
2 Spans of 210 feet,	420.0 "
2 Spans of 227 feet,	454.0 "
1 Span of 370 feet,	370.0 "
6 Spans of 245.5 feet,	1473.0 "
1 Span of 400 feet,	400.0 "
3 Spans of 180 feet,	540.0 "
1 Span of 149.6 feet,	149.6 "
1 Span of 100 feet,	100.0 "
1 Span in Indiana abutment,	32.5 "
Total length,	5294.0 feet.

The superstructure is placed below the grade line, with the exception of the channel spans, which are raised in order to provide room for the passage of boats. The height of the track at these points is 101½ feet above low water, and 50½ feet above high water; this elevation being obtained by grades of 76 feet per mile from either shore to the channel spans, the bridge between the channels being level.

The general plan of the superstructure is that of Mr. Fink, the chords and post shoes being of cast and the posts of wrought iron. The pivot span over the canal is a triangular truss, entirely of wrought iron. The channel spans, of 370 and 400 feet from centre to centre of piers, being the longest spans yet erected in America, are built upon a modification of the triangular, in which auxiliary trusses are introduced in order to obtain a desirable

length of panel in the primary truss, and at the same time fix the braces at the middle, thus practically reducing their length one half, and enabling them to be made with less material.

The primary truss of the 400 feet span is divided into seven panels of 56′ 7¼″ each, from centre to centre of pins. These panels are subdivided into four parts of 14′ 1$\frac{5}{16}$″ each, the top chord being supported at these intervals by posts, and the lower chord suspended by vertical rods at the same distances. Each side truss is double, the distance from centre to centre of the two trusses forming a pair being 41″, and the width from centre to centre of pairs 25′ 7″. The connection between the two trusses in a pair, which is effected by bolts and struts, was not made until after each had assumed the deflection proper to its own weight, thus avoiding any chance of undue strains from inaccuracies in workmanship. No perceptible difference was observed in the curves of the four trusses before the connection.

The top chords are 14″ in external diameter, and reduced in section from the centre to the ends, according to the reduction of the strains, the maximum thickness being 1½ inches, and the minimum 1 inch. The braces are of Phœnix columns, and vary in diameter from 5½ to 17 inches, and in section from 5.7 to 60 square inches. The wrought iron bars and rods are of American bridge-iron. The track stringers consist of four pieces of white pine, 8 × 16 each, and are supported by trussed 12 inch I beams.

The details of the 370 feet span are similar to the above, the general arrangement, however, being different, the primary truss having six panels of 61′ 4″ each.

The entire superstructure of the bridge is proportioned for a moving load of 2600 pounds per lineal foot. The chords have with this load, by Hodgkinson's formula, a factor of 7 for safety, the posts and braces factors from 5 to 6, and the wrought iron proper a strain of from 7000 to 12,500 pounds per inch, according to its position and duty. In the channel spans the calculated maximum strain on the bottom chords is 12,000 pounds, in the ties from 10,000 to 12,000, and in the suspension rods and small

truss bars 7000 pounds per inch. The counterbracing required to resist the effect of a moving load is very light, on account of the great weight of the superstructure as compared with that of the load. This is provided for by wrought iron straps connecting the tops of the braces with the top chord. The weight per foot of the 400 feet span, including the floor-beams and pier-bearings, but excluding track-stringers, cross-ties, track and foot-walks, is 3502 pounds, and without the pier plates, rollers, etc., 3378 pounds per foot. The total weight per foot, including everything, is 4162 pounds. The total weight per foot of the 370 feet span, including everything, is 3664 pounds.

The deflection of the 400 feet span under four engines, weighing 160 tons, was $1\frac{1}{16}$ inches. The 370 feet span, with the same load, deflected $1\frac{1}{8}$ inches; a 245 feet span $1\frac{3}{4}$; a 180 feet span $1\frac{3}{4}$; and a 149 feet span $1\frac{5}{8}$ inches; the original camber in all cases returning upon the removal of the load.

The camber for the 400 and the 370 feet span was put in by making each part longer or shorter than the calculated length by the amount which it would compress or extend under the influence of the maximum load. On this supposition, when the span is fully loaded, the camber should disappear. This result was very nearly reached, the camber of the 400 feet span being $2\frac{1}{2}$ inches when light, and that of the 370 feet span $3\frac{1}{4}$ inches, a small margin being allowed in both to insure a camber in the case of irregularity in work. The elongation of the bottom chord of the 400 feet span, under a train of loaded cars, is, by actual measurement, $\frac{9}{16}$ of an inch.

The camber for the Fink trusses was put in by calculating the length of the chains for a length of post at any point less than the true length by the ordinate at that point.

The scaffolding for the 400 feet span was erected on five large cribs, each 14 feet wide, 50 feet long, and 10 feet high, filled with stone, and bolted to the rock bottom of the river. The first of these cribs was sunk 57 feet from the pier, the next 114 feet from the first, leaving an opening for navigation, while the third, fourth,

and fifth divided the remaining space into openings of about 57 feet each. On the five cribs trestles were erected, and connected at the top, except the long span, by single-post trusses. For the long span, leaning trestles were erected from the cribs on each side at such an angle as to be connected at the tops by the same length of truss as spanned the short openings. Upon the frames thus erected to the grade line the top trestle was built, 50 feet high, and the iron-work put together. The 400 feet span was raised in 21 days, and the 370 feet span in 15 days.

The St. Charles Bridge.

This fine structure crosses the Missouri River at St. Charles, twenty miles from St. Louis. It consists of three spans of double triangular truss, 322 feet each, four spans of Fink truss, 304 feet each, two spans of 64 feet each, one of 48 feet, and 4318 feet of wrought iron trestle-work — making a total length of 6676 feet. These are the longest spans yet built upon Mr. Fink's plan, and the whole work is the longest in the United States. The triangular trusses consist of two systems overlapping by half a panel, making a trellis, with cast iron upper chords, wrought iron keystone columns with wrought iron feet and heads webbed out so as to distribute the weight over 2½ feet in length of the pins on which they rest. All of the posts and ties are pin-jointed, in both upper and lower chords, thus making the connections on the centre line of strains, and while holding the truss firmly at the joints, allowing the greatest freedom of action under changes of temperature and moving loads. The counter-strains for a few panels on each side of the centre are provided for by stiffening the middle ties, and giving the braces a tensile connection. The end posts are inclined, and have both a rocker and a roller action at the bearings. The Fink deck-spans are proportioned to carry 2250 pounds per foot, and the Triangular spans to carry 2400 pounds. The weight of a 304 feet Fink span is 680,000 pounds, and that of a 322 feet Triangular span 788,000 pounds. The cast

iron chords in the Fink span are proportioned for a compression of 12,000 pounds per inch, the main chain for 12,000 pounds of tension per inch, the quarter chain for 11,000 pounds, the eighth and sixteenth chain for 10,000 pounds, and the wrought iron Phœnix columns for 6500 pounds per inch.

The river not allowing of false-work, temporary piers were erected, resting on piles, and surrounded by cribs, 18 feet wide and 50 feet long, filled with stone. There were three of these piers in each span, and on these were placed bodily, by floating derricks, Howe trusses of 80 feet reach, and upon these trusses the false-work proper was placed, the top being 121 feet above the water.

The above work is one of a very large number of exceedingly well arranged and thoroughly made structures by the Baltimore Bridge Company, among which may also be mentioned the Catawba Bridge, upon the Charlotte and South Carolina Railway, having nine spans of 125 feet each; the High Bridge, upon the Atlantic, Mississippi, and Ohio Railway, having 21 spans of 112 feet each; and the Rock Island Bridge, upon the Chicago, Rock Island, and Pacific Railway, 1840 feet in length.

Pennsylvania Railroad Bridges.

Plate XVII. represents the Mount Union Bridge, over the Juniata River, upon the Pennsylvania Railroad, designed by Messrs. Joseph M. Wilson and Henry Pettit, engineers, and built by the Keystone Bridge Company of Pittsburgh. In this work the compression members are vertical, while the ties and counterties are inclined. Under a uniform load the vertical members sustain a compressive strain, and the inclined members tension; not necessarily the maximum strains however, which, except for the end members, occur under certain conditions of the variable load. Under the action of the variable load certain inclined members sustain compression, and certain vertical members tension; but the amount of such strain is not very great, especially

in large spans where the dead weight of the bridge plays an important part, and it is easily provided for by constructing such vertical members to resist tension, and such inclined members to resist compression, the vertical members requiring no more material than needed under maximum compressive strains, and the inclined members only a small amount more than required for maximum tensile strains. To farther induce economy of construction and simplicity of details, the angle of the inclined members from the vertical is taken at about 45 degrees, and a single intersection system is adopted, thus massing the material as much as possible. In a bridge of long span, however, by this arrangement the triangles become quite large, and the inclined braces long, while for a deck bridge, the upper chord is left for too great a distance in each panel unsupported.

A secondary system has, therefore, been introduced, consisting of a light vertical post and an inclined panel stiffener, the two connecting with the inclined main carrying brace at its middle point, thus shortening and stiffening it for compression, and at the same time effectually trussing the upper chord for the length of one panel. In case of a through bridge, the lower chord would be trussed instead of the upper chord.

The structure is designed as a double track deck bridge of three trusses, the latter being so placed that when both tracks are loaded, each truss carries one third of the total load. It is constructed entirely of wrought iron, except certain connecting pins, which are of steel, and the rollers under the bolster blocks at the ends of the trusses, which are chilled castings. The different members are joined together by pin connections throughout.

The general dimensions of the work are as follows: One span of 121′ 6″, three spans of 123′ 6″ each, and one span over the canal 125′ 4″; the above lengths being from centre to centre of end posts. There are three trusses, 9′ 6″ apart from centre to centre, the bridge being for a double track. Each truss has 8 main panels and 16 sub-panels. The height of truss from centre to centre of chord pins is 15′ 8″. The extra length given to the

16

upper chord over the lower one for camber is $\frac{1}{16}$ inch in each sub-panel, or one inch in each span.

The ultimate strain per square inch for tension is taken at 60,000 pounds, and for compression in the case of short prisms, 36,000 pounds, Gordon's formulæ being used in the computation of columns, and a factor of safety of 6 adopted throughout the structure.

The upper chord is formed of channel and deck beams, connected together at the top by a rolled plate riveted on, an increase of section being given to the chord towards the centre of the bridge by the addition of rolled plates on top, and also of thickening plates to the sides of the webs of the deck beams.

The lower chord is composed of links, 5 to 7 inches deep, and of varying widths, having upset heads at the ends drilled for connecting pins, the upset heads being $\frac{1}{4}$ inch thicker than the body of the link.

The vertical posts of the primary system are constructed of I beams and channel bars, connected together and stiffened, as shown in the plate.

The main carrying links are flat bars, 6 inches deep, and of varying widths, and arranged in pairs ; those towards the centre, which have to resist compression under the action of the variable load being braced by internal diagonal bracing, and joined by rivets with distance ferrules. The ends are upset and drilled for connecting pins, the upset heads being $\frac{1}{4}$ inch thicker than the main body of the link. In the centre of their lengths, where the intermediate connecting pins pass, the braces have thickening plates riveted on so that the proper section shall be maintained.

The vertical posts of the secondary system are composed of two light channel bars, braced by internal diagonal bracing, and joined by rivets and ferrules. The inclined panel stiffeners are formed of two rolled links, 3 inches wide, bulged out, and stiffened by rivets and distance ferrules.

The upper and lower chord connecting pins of the primary system are all 4 inches diameter ; the upper chord connecting pins of the secondary system are $1\frac{1}{4}$ inches in diameter, and the

intermediate connecting pins in centre of length of main carrying braces are 1½ inches in diameter, and of steel. Where necessary the ends of the pins project out, and are planed off flat on the sides, to afford connections for the lateral and diagonal bracing and struts. The lateral struts are composed of two pieces of rolled iron, bulged and stiffened by rivets and ferrules. The lateral and diagonal bracing is of round rods, with sleeve nuts for adjustment. The upper lateral bracing is between the centre and side trusses, and in sub-panel lengths. The lower lateral bracing has the rods pass from outer truss to outer truss, above and below the lower chord of the centre truss. The struts are between the centre and side trusses, as in the upper chord. Diagonal bracing is placed at every post in the primary system, and in the two central panels only of the sub-system. The latter is merely useful for stiffening the main carrying braces, and is formed of light rods, the struts for the intermediate pins being made of two pieces of light T iron, bulged and stiffened by rivets and ferrules.

The bolster blocks and pier plates are of wrought iron, constructed under the Wilson patent, and the former have hinge connections with the trusses, allowing of adjustment of position. Each truss is fixed at one end, the bearing blocks at the fixed end, and the rollers at the other being chilled castings. Thickening washers are used on all connecting pins wherever necessary to make all joints close and tight.

Plate XVIII. illustrates Bridge No. 5, over the Little Juniata, upon the Pennsylvania Railroad, designed by Messrs. Wilson and Pettit, and built by the Keystone Bridge Company. The superstructure is in two spans, of three trusses each, arranged as a half through bridge for two tracks, the roadway resting upon rolled I beams, supported by the lower chords. The trusses are constructed upon the single intersection triangular system, with vertical carrying rods and inclined lateral stays in each panel; the different members being held together at their intersections by connecting pins. The following are the general dimensions of the structure. Distance from centre to centre of pins, measured on

the lower chord, 82′ 6″. Number of panels in each truss, 6. Length of each panel 13′ 9″. Height of truss, centre to centre of chord pins, 8′ 3″. Width between trusses, centre to centre, 14′. Width of bridge seat on abutments, 4′ 3″; on piers, 6′ 6″. In proportioning the different parts of the structure the variable load was assumed at $1\frac{1}{2}$ tons per foot lineal of each track, the middle truss being calculated for both tracks loaded. The ultimate strain for wrought iron was taken at 30 tons per square inch of section for tension, and 18 tons for compression, in the case of short prisms, Gordon's formulæ being used in computation of columns. The factor of safety adopted throughout was 6.

The upper chord, as will be seen by reference to the Plate, is composed of 9 inch deck beams, united on top by rolled plates. The section of these plates is increased towards the centre of the bridge, to provide the proper area at the various points of the chord, and thickening pieces are also introduced on the sides of the webs of the deck beams for the same purpose. The lower chord consists of links 7 inches deep, and of varying widths, upset at the ends, with eyes for $4\frac{1}{2}$ inch pins. The main braces are formed of links, having eyes at the ends for connecting pins, and arranged in sets, those sets which are required to resist compression as well as tension being bowed out and connected together by rivets, with distance ferrules between them. The vertical carrying rods have an eye at the upper end fitting on to the upper chord pin, and at the lower end each passes through and sustains a cast iron shoe which supports the lower chord. The lower chord connecting pins project out where necessary, and the projections are planed down on the sides to flat surfaces, so as to furnish connections for the lateral struts and bracing. The lateral struts are formed of two pieces of rolled iron, $4\frac{1}{2}$ by $\frac{3}{4}$ inches, bulged and connected by rivets and ferrules. The lateral stays are formed of the same section of iron. The lateral bracing consists of round rods, each rod having a sleeve nut for adjustment. The inclined end posts are of cast iron, with cast iron bolsters and pier and abutment plates, one end of each span being provided with rollers, and the other end fixed. The rollers are

chilled castings. The floor is supported by 9 inch rolled I beams, weighing 30 pounds to the yard, and placed 2 feet 3 inches apart. The track stringers are 6 by 12 inches, white oak, notched $\frac{1}{2}$ inch on the floor beams, and fastened by bolts to the same.

Bollman's Bridge.

Fig. 1, Plate XIX., shows the general plan of Bollman's Bridge, of which numerous examples may be seen upon our railways, the finest, perhaps, being that across the Potomac, at Harper's Ferry. Each post is supported by an independent pair of bars running from its foot to the top of each abutment tower. Fig. 2 is an elevation of the end, showing the abutment tower. Fig. 3 is a plan of the flooring, with the lateral bracing. Fig. 4 is a plan of a section of the upper chord, showing the connection of the lateral brace. Fig. 5 is an end view of the top chord over the post. Fig. 6, a side elevation of the same, with one section (Fig. 7) removed, to show the tenon. Figs. 8 and 9 show side and transverse views of the compensating link, in which the main suspenders are seen to take hold of the upper pin, while the panel rods, the post, and the floor are attached to the lower one; the two being connected by the link. By this arrangement the upper pin is enabled to change its position in accordance with the unequal expansion or extension of the long and short rods connected with it, without straining the posts or disturbing the action of the panel rods. The bridge is adjusted by screwing the nuts on the bottom of the short rod which connects the floor and post with the lower pin. One man with a wrench 18 inches long can raise any panel of a 150 feet bridge. Figs. 10 and 11 show the cast iron chair between the floor beam and the floor girder.

As commonly built, the top chord and posts are of cast iron, and the tension bars of wrought iron; though, of course, wrought iron compression members are as applicable here as in other bridges. Besides the main suspension systems, the diagonal panel rods are introduced to counteract the effect of a partial load upon the bridge; they are, moreover, made large enough to carry the

load upon one panel. The feet of the posts are maintained at a proper distance by struts either of wood or cast iron. The cast iron chord is octagonal without and circular within, cast in lengths, and connected over the post with a tenon and socket, the former being turned and the latter bored, the whole thus forming a continuous pipe. The tenons are very slightly rounded at the ends, to allow a small angular movement without breaking the joint. The abutment ends of the chord are cast, as in the plate, to receive the main suspenders, which latter pass up each side of the chord, and connect behind it with eye bolts. The value of the wrought iron in the tension members is taken by Mr. Bollman at 10,000 pounds per square inch throughout, except for the link and suspension bolts, for which the value assumed is 6000 pounds. Placing 20,000 pounds at the foot of each post, the strains and size of the bars in the example given in Plate XIX. will be as below, the bridge and load weighing 3360 pounds per foot, or 1680 pounds for each side truss.

No. of Rod.	Vertical Strain.	Direct Strain.	Value of Iron.	Section of each of two Bars.	Size of Bar.
1	20.000 $\times \frac{7}{8}$ 17.500	17.500 $\times \frac{24.0}{18.5} =$ 22,700	10,000	1.14	$\frac{7}{8} \times 1\frac{3}{8}$
2	20,000 $\times \frac{6}{8}$ 15 000	15.000 $\times \frac{32.0}{18.5} =$ 25.946	10,000	1.30	$\frac{7}{8} \times 1\frac{1}{2}$
3	20.000 $\times \frac{5}{8}$ 12.500	12.500 $\times \frac{42.0}{18.5} =$ 28.378	10.000	1.42	$\frac{7}{8} \times 1\frac{3}{4}$
4	20.000 $\times \frac{4}{8}$ 10.000	10.000 $\times \frac{53.25}{18.5} =$ 28.783	10,000	1.44	$\frac{7}{8} \times 1\frac{3}{4}$
5	20,000 $\times \frac{3}{8}$ 7,500	7.500 $\times \frac{64.5}{18.5} =$ 26.149	10,000	1.31	$\frac{7}{8} \times 1\frac{1}{2}$
6	20.000 $\times \frac{2}{8}$ 5.000	5.000 $\times \frac{75.0}{18.5} =$ 20.270	10.000	1.01	$\frac{7}{8} \times 1\frac{1}{4}$
7	20.000 $\times \frac{1}{8}$ 2.500	2.500 $\times \frac{87.5}{18.5} =$ 11,824	10,000	0.59	$\frac{3}{4} \times \frac{7}{8}$

The panel rods each bear half of 20,000, or 10,000 pounds, which, multiplied by $\frac{23}{19}$, gives 12,105 as the direct strain, which, at 10,000 pounds, requires an area of 1.21 inches, or one bar 1 $\times$ 1$\frac{1}{4}$. The suspension bolts bear each (there being two) 10,000 pounds, and at 6,000 pounds per inch require 1.66 inches, obtained by one 1$\frac{1}{2}$ inch round rod. The links bear each 20,000 $\div$ 4, or 5000, which,

at 6000 per inch, requires an area of 0.83 inches, obtained by a bar $\frac{7}{8} \times 1$ inch. The upper diagonals have a diameter of 1 inch, and the lower ones of $1\frac{1}{4}$ inches.

The Phœnixville Bridges.

The bridges built at Phœnixville, by Messrs. Clarke, Reeves, and Company, have been very widely adopted throughout the United States, and for excellence of design and thoroughness of workmanship are unsurpassed. The general arrangement of the truss for an overgrade bridge is that shown in Fig. 108, page 248. The arrangement for an undergrade bridge is similar to that shown in Fig. 103, page 222, except that Messrs. Clarke and Reeves counterbrace the whole length of the truss. The details for both overgrade and undergrade bridges are illustrated in Plate XX., where Figs. 1 and 2 show the connection at X, in Fig. 108; Figs. 3 and 4, the foot of the main brace and its bearing on the masonry; Figs. 5 and 6, the side elevation and section at H., Fig. 108; and Figs. 7 and 8 the same at G. Fig. 9 is the connection for the top chord and end post for an undergrade bridge. Figs. 10 and 11 are side and cross views at the top of the intermediate posts, and Figs. 12 and 13 the same at the bottom. The upper chords and the posts are wrought iron Phœnix columns. The ties and lower chords are flat eye-bars. All of the connections are upon pins, thus bringing the strains upon the centre lines of the various members.

The bridge shown in outline in Fig. 108 is 177′ 5″ from centre to centre of end pins; the panels, 13 in number, are 13′ 6″, except the end ones, which are 14′ $5\frac{1}{2}$″; the height from centre to centre is 27′; the width from centre to centre of top chords, 16′. The pins at the chord and tie connections are 3 inches in diameter. The transverse floor beams consist of two 15 inch rolled girders. The estimated weight of bridge and track is 1140 pounds per foot; the rolling load 2500; total 3640, or $\frac{3640}{2} \times 13\frac{1}{2}$ $= 24,570$ pounds on each panel of each truss, except in the case of the 1st, or vertical, and the 5th and 6th sets, for which a rolling

load of 3000 pounds per foot is taken, making a special panel load for those cases of 4140 pounds per foot. With these data the strains upon the several members may easily be estimated. The rod A B, Fig. 108, sustains the two half panels adjacent to

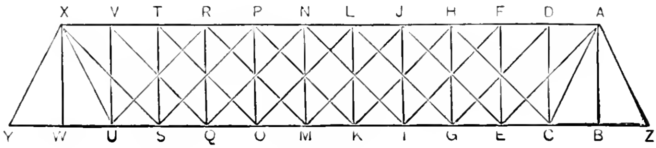


Fig. 108.

the foot of it. A C sustains the two half panels adjacent to its foot, plus the two half panels adjacent to the point G, plus the two adjacent to the point K, being in all 3 panels. The rod A E sustains in the same way the half panels adjacent to E and I; D G the half panels at G and K; F I the half panels at I, and H K the half panels at K. Thus A B supports 1 panel, A C 3 panels, A E and D G 2 panels, F I and H K 1 panel each, and the weight of the several panels sustained by each rod is to be increased, as before remarked, for the inclination of the rod. Beyond the centre the rods are strained only by partial moving loads. Of the two half panels adjacent to U, $\frac{2}{13}$ go to the abutment Z by means of the rod U R. Of the load at S, $\frac{3}{13}$ go to Z by means of S P. Of the load at Q, $\frac{4}{13}$ go to Z, and of the load at O, $\frac{5}{13}$. Thus U R is strained by $\frac{2}{13}$ of the weight of a panel, S P by $\frac{3}{13}$, Q N by the $\frac{2}{13}$ thrown upon it by U R, plus its own $\frac{4}{13}$, or in all $\frac{6}{13}$, and in the same way O L by $\frac{3}{13} + \frac{5}{13}$, and M J by $\frac{2}{13} + \frac{4}{13} + \frac{6}{13}$. And these weights increased for the inclination give the strains upon the several rods. The strains upon the chords are obtained by the method already noticed in Chapter X. The end posts, of course, sustain the total weight of the bridge and load, except the half panels next the abutments.

The Detroit Bridge and Iron Company's Work.

Fig. 1, Plate XXI., represents a half span of the longer reaches of the railway bridges across the Mississippi River at Burlington,

Iowa, and at Quincy, Ill., built at the Detroit Bridge and Iron Works. The span is 250 feet, the height from centre to centre 26 feet, and the width (single track) 14 feet clear, inside. The general plan is the double Whipple, as already noticed. The weight of the bridge and load is taken at 3960 pounds per lineal foot, and the factor of safety used is 6. The strains figured on the plate are for one side truss. Fig. 2 shows a top plan, with lateral bracing. Fig. 3 shows the detail at the top and bottom of the end post, and also a side elevation of the connections at the feet of the first and second suspension bars. Fig. 4 shows a section of the top chord, and transverse view of post and lower connection. The top chord is of cast iron, octagonal without and circular within, 14 inches exterior diameter, uniform throughout, but varying in thickness according to the compression upon it. The posts are wrought iron composite columns, as shown in the sketches. Fig. 5 shows the side elevation of a 60 feet wrought iron trussed girder, the top chord being made of rolled beams, and the posts of cast iron. Fig. 6 is a plan of the top chords, showing the lateral bracing. Figs. 7 and 8 are enlarged transverse sectional views. The strains given upon Fig. 5 are for a single truss, the load being assumed at 3000 pounds per foot, and the factor of safety being taken at 6. The width is 6 feet from centre to centre. For a 30 feet span, trussed with a single post 4 feet high, a chord the same size is employed, viz., 3 × 12; the compression on the top chord being 47,000 pounds, the tension upon the rods 48,000, and the load upon the post 25,000 pounds.

The Burlington Bridge.

This is among the best works built by the Detroit Bridge and Iron Company. It has one span of 175 feet, one of 200, six of 250, and a draw of 360 feet; making, in all, 2235 feet. The general plan is the Trapezoidal, or Whipple. Each span has panels of about 12 feet in length. The tension rods are in pairs, with a run of two panels. The upper chords and pedestals, of the fixed spans, are of cast iron, and all the other parts of wrought

iron. The draw spans are substantially of wrought iron throughout. The floor is carried on wrought iron beams, suspended in pairs, under the feet of the posts.

The main and counter ties, the links composing the lower chords, and the bolts sustaining the floor beams are all coupled on pins, which pass through the feet of the vertical posts. The upper ends of main and counter ties pass through the top chords. and are secured thereto by screws and nuts. The wrought iron used was of the best and toughest quality. Before being manufactured it was tested as follows: From each lot of one hundred bars, as they came from the rolling-mill, five were indiscriminately selected, and tested to absolute rupture by tensile strain. If any one of the samples broke under a less strain than 60,000 pounds per square inch of sectional area, the lot was rejected. After being manufactured, and before being placed in the bridge, every bar was submitted to an actual tensile strain of 20,000 pounds per square inch, and, while under such strain, was struck smartly several times with a hammer. Any bar was condemned which, under this treatment, showed permanent set, or any evidence of imperfection.

The iron used for tension was very uniform in quality. The mean ultimate strength of all the pieces tested was about 64,000 pounds per square inch; some of them developing a strength of over 80,000 pounds per square inch of original area. The average stretch of ruptured bars was about 20 per cent. of their original length, and the ultimate strength per square inch of fractured section was over 100,000 pounds. All hollow cast iron chord pieces were drilled and callipered, to detect any inequality in the thickness of the sides. The iron used was mainly from the Lake Superior ores, and gave excellent results in compressive resistance. The coupling-pins were forged from selected scrap, and were finished in the lathe. to fit perfectly the drilled eyes.

The 250 feet spans were adjusted to a camber of $2\frac{1}{2}$ inches, the 200 feet spans to a camber of 2 inches, and the 175 feet span to a camber of $1\frac{1}{2}$ inches. The deflections under the impositions of a maximum load were about equal to the above cambers, respec-

tively, it being intended that, when fully loaded, the bridge should be horizontal, thus bringing the joints in the top chords into full bearing over their entire area.

Of course this deflection could have been lessened, or entirely obviated, by screwing down the counter ties when the bridge was fully loaded. In a bridge so easily adjustable as are these structures, the amount of deflection is entirely under control, and can be diminished, or reduced to zero, at pleasure. But it was deemed best by the constructors to leave the bridges to their natural adaptation. The way to prevent deflection is by a permanent load, produced either by an actually imposed weight, or by heavy strains on the counter ties. Such permanent loading was not considered desirable, and the deflection may therefore be considered to represent the results of the natural elasticity of the iron under the effect of the moving load.

The distribution of the weight of material in the 250 feet spans is as below: —

Items.	Tons Cast Iron.	Tons Wrought Iron.	Total.
Top Chords,	39	...	39
Lower Chords,	...	38	38
Posts,	14	27	41
Main and Counter-Ties,	...	23	23
Lateral System,	...	8	8
Floor,	3	17	20
Pins,	...	3	3
Wall Plates and Rollers,	4	...	4
Total without Track,	60	116	176
Track Timbers and Track,	...	...	39
Total weight of one Span including Track,	...	...	215

The following trials were made upon the completed structure: The first test was on a span of 250 feet, with two heavy engines and four cars, two in front and two in rear of the engines, loaded heavily with stone and iron. The length of this train was about 220 feet, and the weight about 180 tons. This train being brought to a stop caused a deflection in the centre of the span, on the side of the bridge, of $2\frac{3}{8}$ inches, and in the centre of the floor-beam of $2\frac{7}{16}$ inches, returning, on the removal of the load, to within $\frac{1}{16}$ of an inch of the original camber.

The same load moving from four to six miles per hour caused a deflection on the side of the bridge of $2\frac{7}{16}$ inches, and in the centre of the floor beam of $2\frac{1}{2}$ inches; returning, on removal of the load, to within $\frac{1}{16}$ of an inch of the original camber.

The same load moving from ten to twelve miles an hour caused a deflection on the side of the bridge of $2\frac{9}{16}$ inches, and in the centre of floor-beam of $2\frac{5}{8}$ inches; returning, on removal of load, to within $\frac{1}{16}$ of an inch of the original camber.

The contract for the above work was let in May, 1867, and the bridge was opened in August, 1868. A gang of thirty trained men were able to put up a 250 feet span in about a week.

The Keystone Bridges.

The bridges built by the Keystone Bridge Company have been very widely adopted, and are, in every sense, first-class structures. The general plan is the Whipple, with double intersections, as already shown. The top chord is formed of rolled irons riveted to a top and bottom plate. The end post is shown in section by Fig. 109, and the intermediate posts by Fig. 110. The columns are swelled at the middle by increasing the length of the rivet and the thimble, as shown in the pivot bridge, represented in Plate XXVI.

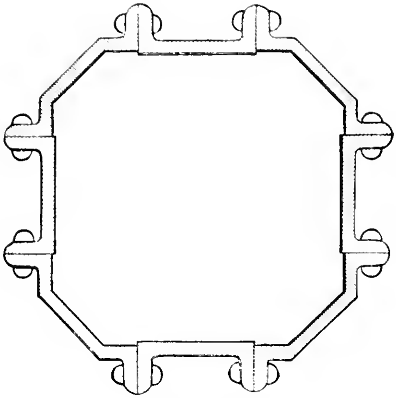


Fig. 109.

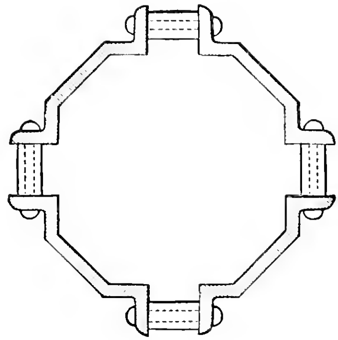


Fig. 110.

THE CANESTOTA BRIDGE.

Fig. 1, Plate XXII., represents a wrought iron lattice bridge upon the New York Central Railroad, over the Erie Canal, near Canestota Station, built by Mr. Charles Hilton. The span is 125 feet, the bridge being upon an angle of 31 degrees. The height is 19′ 4″. The panels are 9′ 3″, and the width 14′ 7″, for each line of rails, the bridge being for a double track. Fig. 2 shows the top chord, the end post, and the connection with the struts and ties, with the arrangement of the rivets, and the same for the lower chord. A is a section of the end posts at B, and C is the intersection of the diagonals. Fig. 4 is a transverse section through the top, showing the connection of the ties and braces, and the cross bracing, or stiffening of the latter. Fig 5 shows the lower chord, the cross girder D, and the longitudinal floor girder G. Fig. 6 is an enlarged view of the cross girder D at the middle truss, and Fig. 7 at the side truss. Fig. 8 is a section through the cross girder D, showing the connection of the longitudinal floor girder G, a part of which is removed to show the chord plate E, the angle iron F, and brace H, beyond. Fig. 9

is an under side view of the lower chord splice, showing the disposition of rivets, the cross girder D, and diagonal I. Fig. 10 is an elevation of the same, showing the splice of the vertical plate. The method of computation for the strains upon the several members, as given by Mr Hilton, is as below:—

Span, 125 feet; rise, 19′ 4″; panels, 14 of 9′ 3″ each; angle of braces, 45°. The whole load is assumed to be at the floor. The girder consists, as seen in Fig. 111, of four separate systems

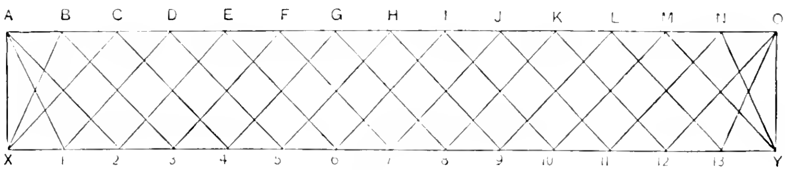


Fig. 111.

of triangulation, and each system is treated independently of the others. Thus, a load placed at any apex is transmitted to the abutments by the members of the system to which that apex belongs, without direct aid from the members of any other system. The members of these systems are alternately braces and ties, thus forming a triangular truss; and any strain or force passing through a diagonal towards either abutment produces, in ascending from the lower to the upper chord, tension, and in descending from the upper to the lower chord, compression, upon the diagonal.

If one or more apices on each side of any diagonal are loaded, the proportions of the loads on either side passing through the diagonal to the opposite abutments counteract and neutralize each other to the extent of the smaller of the two forces, and the excess of one over the other is the measure of the actual strain upon the member.

It is evident from this, that the maximum strain upon any diagonal will occur when all the apices on the side towards the most distant abutment are loaded to the greatest possible extent, and those on the opposite side have the smallest possible load,

which is, of course, the weight of the superstructure itself; for, in such case, the difference between the opposing strains will be the greatest possible.

The chords are, of course, common to all the systems, and receive and transmit the accumulated horizontal components of the strains on all the diagonals. The maximum strain upon the chords occurs when the girder is loaded throughout with the greatest possible load.

The greatest moving load is assumed to be $1\frac{1}{2}$ tons per lineal foot of track, and for a double track bridge 3 tons (2000 pounds per ton). Each of the outer trusses will of course carry $\frac{1}{4}$ of the above, or $\frac{3}{4}$ of a ton per foot. The dead load, or weight of the superstructure itself, is taken at $\frac{1}{2}$ ton per foot of single track, or $\frac{1}{4}$ ton per foot on each girder. The panels being 9′ 3″ long, we have at each apex of the girder a moving, or live load, of 6.94 tons, and a dead load of 2.31 tons, or in all 9.25 tons. To find the maximum strain which the diagonal A 1 is liable to sustain, we must assume the girder to be loaded with its maximum load from 1 to 13. The apices of the system of which diagonal A 1 is a member, are 1, 5, 9, 13, and the porportions of the loads at these apices that are transmitted through diagonal A 1 to abutment X are respectively $\frac{13}{14}$, $\frac{9}{14}$, $\frac{5}{14}$, and $\frac{1}{14}$; and if we call the total live and dead load at each apex l, and the angle which the diagonal makes with the vertical δ, we shall have for the strain on diagonal A 1, $\frac{13+9+5+1}{14}\,l \sec.\delta$, or $28\frac{l}{14} \sec.\delta$. By the same method we find the strain on A 2 to be $24\frac{l}{14} \sec.\delta$; on B 3, $21\frac{l}{14} \sec.\delta$; on C 4, $18\frac{l}{14} \sec.\delta$; and on D 5, $15\frac{l}{14} \sec.\delta$; but from the last must be deducted the proper proportion of the dead load at apex 1, which, by its tendency to pass through diagonal D 5 to abutment Y, counteracts and neutralizes its equivalent of the opposite force; therefore, calling the dead load at each apex l', the exact strain on diagonal D 5 will be $15\frac{l}{14} \sec.\delta - 1\frac{l''}{14} \sec.\delta$. Calling the coefficients of $\frac{l}{14} \sec.\delta$ and $\frac{l''}{14} \sec.\delta$, n and n' respectively, and $\frac{l}{14}$ and $\frac{l''}{14}$ w and w' respectively, the above formula becomes in general $n\,w \sec.\delta - n'\,w' \sec.\delta$. Now w and w' are constant, as also is

sec. δ, except for the end diagonals, and have to be ascertained but once.

The following tables, showing the strains upon the several diagonals, are computed by this formula. The first column contains the designations of the diagonals; the second column the factors, $n \times w$, sec. δ; the third column the factors $n' \times w'$, sec. δ; and the fourth column contains the actual strains upon the diagonals in tons of 2000 pounds. The fifth column shows the net sectional area in inches required in the diagonals, allowing 5 tons per square inch in tension, and 3 tons per square inch in compression.

TABLE I. — COMPUTATION OF TENSILE STRAINS ON DIAGONALS.

Tie.	$n \times w$ Sec. δ.	$n' \times w'$ Sec. δ.	Strain.	Inches.
A 1	28 × 0.7324	. . . . . .	20.69	4.14
A 2	24 × 0.9144	. . . . . .	22.42	4.48
B 3	21 × "	. . . . . .	19.61	3.92
C 4	18 × "	. . . . . .	16.81	3.36
D 5	15 × "	— 1 × 0.2286	13.78	2.76
E 6	12 × "	— 2 × "	10.74	2.15
F 7	10 × "	— 3 × "	8.64	1.73
G 8	8 × "	— 4 × "	6.54	1.31
H 9	6 × "	— 6 × "	4.21	0.84
I 10	4 × "	— 8 × "	1.87	0.37
J 11	3 × "	— 10 × "	0.47	0.09

TABLE II.—COMPUTATION OF COMPRESSIVE STRAINS ON DIAGONALS.

Diagonals.	$n \times w$ Sec. δ.	$n' \times w'$ Sec. δ.	Strain.	Inches.
A X	$(28 + 24) \times 0.661$	. . . . . .	34.37	11.46
B X	21×0.7324	. . . . . .	15.52	5.17
C X	18×0.9144	. . . . . .	16.81	5.60
D 1	$15 \times$ "	$- 1 \times 0.2286$	13.78	4.60
E 2	$12 \times$ "	$- 2 \times$ "	10.74	3.58
F 3	$10 \times$ "	$- 3 \times$ "	8.64	2.88
G 4	$8 \times$ "	$- 4 \times$ "	6.54	2.51
H 5	$6 \times$ "	$- 6 \times$ "	4.21	1.40
I 6	$4 \times$ "	$- 8 \times$ "	1.87	0.62
J 7	$3 \times$ "	$- 10 \times$ "	0.47	0.16

In the preceding tables the computations are carried forward as far as the formula will give a positive result; thus, in Table I., the formula for diagonal K 12 would give for the strain —0.93; which shows that under no circumstances can diagonal K 12 be subjected to a tensile strain. In Table No. II., the formula would give for diagonal K 8 the same negative result; which shows that that diagonal can never be subjected to a compressive strain.

It has been assumed in the preceding computations that the girder was loaded with its maximum moving load from abutment Y to all the apices from 1 to 11, successively; but if we assume the girder to be loaded from abutment X to all the apices from 13 to 3, successively, we shall find some of the diagonals subjected to strains opposite in character to those found for them in Tables I. and II. Thus, diagonals J 11, I 10, and H 9, which in Table I. appear subjected to tensile strains, will now appear under com-

pression equal in amount to diagonals F 3, G 4, and H 5 in Table II., to which diagonals J 11, I 10, and H 9 severally correspond relatively to the abutments and middle of the girder. Thus it appears, that in the passage of every load equal to the one assumed, and covering the whole length of the bridge, certain diagonals will be subjected alternately to tension and compression. The diagonals that will be in this condition, and the amounts of the alternating strains, may be readily ascertained in the following manner: —

Diagonals.	Tensile Strains.	Diagonals.	Compressive Strains.
J 11	0.47	F 3	8.64
I 10	1.87	G 4	6.54
H 9	4.21	H 5	4.21
G 8	6.54	I 6	1.87
F 7	8.64	J 7	0.47

At the top of the first column is placed the designation of the last diagonal in Table I., and opposite, in the second column, is set the tensile strain found for it in said table. On the same line in the third column, is placed the designation of the diagonal corresponding to J 11 on the opposite side of the middle of the girder, which in this case is F 3, and opposite, in the fourth column, is set the compressive strain found for this diagonal in Table II.

The other pairs of corresponding diagonals that are found in Tables I. and II., subjected to different strains, are arranged in the same manner, and the diagonals are taken in their regular order of succession, and continued as far as a diagonal is found, the corresponding diagonal of which, on the opposite side of the middle of the girder, is subjected to a strain of an opposite character.

It is evident that if with a load advancing from Y towards X,

diagonal J 11 is subjected to a tensile strain of 0.47 tons, and F 3 to a compressive strain of 8.64 tons; with a load advancing from X towards Y these strains will be reversed, and diagonal J 11 will be subjected to a compressive strain of 8.64 tons, and F 3 to a tensile strain of 0.47 tons. The same is of course true in regard to I 10 and G 4, H 9 and H 5, G 8 and I 6, F 7 and J 7. As a moving load of uniform weight per foot, and covering the whole length of the bridge, occupies during its passage every position relative to the diagonals that is occupied by a similar load passing over the bridge successively in opposite directions, it follows that each diagonal in the preceding couples will be subjected by the passage of the load to the strains of both kinds as found in Tables I. and II. alternately, and in construction must be proportioned and shaped to resist both strains.

The strains upon the chords are the horizontal components of the strains upon the diagonals, accumulated from each end towards the middle of the girder, where the maximum occurs. The strain added to the chord at each apex between the end and middle of the girder is the sum of the horizontal components of the two diagonals that intersect the chord at that apex. Thus, the strain on X 1, is the sum of the horizontal components of the strains on diagonals B X and C X; and the strain on 1 2, is the strain on X 1, plus the horizontal components of the strain on diagonals A 1, and D 1, and so on. Using the same signs as before, n and n' will have the same value for the same diagonals as in the preceding tables; but sec. δ will become tan. δ; and as the girder is now supposed to be equally loaded over its entire length, w will equal w'. It will be observed that the maximum strains upon the upper and lower chords at the middle, as obtained by this method, are not equal, nor is either of them the same as that given by the well known formula $\frac{W\,l}{8\,d}$; in which W is the total distributed load on the girder, including its own weight, l the length of girder in feet between supports, and d the depth between the centres of the sections of the upper and lower chords. The result given by the last named formula will be found to be the

arithmetical mean between the strains at the middle of the upper and lower chords, as found in the following tables. This discrepancy, or ambiguity, is caused by the diagonals I 6, and G 8, which cross the middle of the girder.

TABLE III. — COMPUTATION OF STRAINS IN LOWER CHORDS.

Panel.	$(n - n') \times w$ Tan. δ.	Increments.	Strains.	Inches.
X 1	(21 × 0.33) + (18 × 0.66)	. . .	18.81	3.76
1 2	(28 × 0.33) + (14 × 0.66)	18.48	37.29	7.46
2 3	(36 — 2) × 0.66	22.44	59.73	11.95
3 4	(31 — 3) × "	18.48	78.21	15.64
4 5	(26 — 4) × "	14.52	92.73	18.55
5 6	(21 — 7) × "	9.24	101.97	20.39
6 7	(16 — 10) × "	3.96	105.93	21.19
7	(10 — 3) × "	4.62	110.55	22.11

TABLE IV. — COMPUTATION OF STRAINS IN UPPER CHORDS.

Panel	$(n - n') \times w$ Tan. δ.	Increments.	Strains.	Inches.
A B	(28 × 0.33) + (24 × 0.66)	. . .	25.08	6.27
B C	(21 × 0.33) + (21 × 0.66)	20.79	45.87	11.47
C D	36 × 0.66	23.76	69.63	17.41
D E	(30 — 2) × 0.66	18.48	88.11	22.03
E F	(24 — 4) × "	13.20	101.31	25.33
F G	(20 — 6) × "	9.24	110.55	27.64
G H	(16 — 8) × "	5.28	115.83	28.96
H	(6 — 6) × "	0.00	115.83	28.96

For the increments for 7, in Table III., and H, in Table IV., we take the horizontal component of but *one* of the diagonals; for if we should take the sum of the two, as elsewhere, we should have the strain on 7 8, instead of 7, and H I instead of H. With the floor attached to the lower chord, the strain on H is the same as on G H and H I; but the maximum strain is at the point 7.

The fifth columns in Tables III. and IV., contain the sectional area required in the chords, allowing five tons tension and four tons compression per square inch. This sectional area must, in all cases, be understood to mean the *net* section after deducting rivet holes for parts in tension, and the *gross* section, without deduction, for parts in compression.

For the middle girder, all the parts, as computed in the preceding tables, are increased by 75 per cent.

The weight of material in this bridge is as follows: —

Chords, ...	66,740	pounds.
Diagonals, ...	52,260	"
End posts, including arches, ...	12,350	"
Floor, ...	48,140	"
Sway bracing, top and bottom, ...	11,440	"
Rivets, ...	17,500	"

Total, 208,430 pounds, or 1579 pounds per lineal foot.

It has been objected to the riveting together of the diagonals at their intersection that the transmission of strains is thus complicated and disturbed, and that the rivets are liable to work loose at these points. The objection is simply a theoretical one, and has no weight whatever in practice. The Canestota Bridge has been in constant use under the heaviest and most rapid traffic for about eight years, but there has never been a loose rivet in the entire structure. "The riveting together of the lattice," says Mr. Latham, "can never add to the longitudinal strain upon it; but by calling into play the resistance of the bars to curvature, adds to the stiffness of the bridge."

The Canestota Bridge was designed and built with great care in all its details, and is well worth a careful study.

Connecticut River Bridge at Warehouse Point.

The iron bridge, designed by James Laurie, Esq., upon the Hartford and New Haven Railway, at the crossing of the Connecticut River near the village of Windsor Locks, and built by William Fairbairn and Company, of Manchester, England, is of the following dimensions and weights: —

1 Span	$177\frac{1}{4}$ feet	$16\frac{3}{4}$ feet high	$10\frac{1}{2}$ feet wide, *c* to *c*,	132 tons.
1 Span	140 feet	$12\frac{1}{2}$ feet high	16 feet wide	91 tons.
12 Spans	$88\frac{1}{2}$ feet	11 feet high	$8\frac{3}{4}$ feet wide	359 tons.
1 Span	$76\frac{3}{4}$ feet	7 feet high	$8\frac{3}{4}$ feet wide	24 tons.
1 Span	43 feet	$3\frac{1}{2}$ feet high	$8\frac{3}{4}$ feet wide	9 tons.
1 Span	$25\frac{1}{2}$ feet	2 feet high	$8\frac{3}{4}$ feet wide	4 tons.

The weight of floor and rails is about 325 pounds per lineal foot. The $25\frac{1}{2}$ and 43 feet spans are boiler plate girders. The remaining spans are the Trapezoidal, or Whipple; the tension bars in the 177 feet span having a run of 3 panels, in the 140 and $88\frac{1}{2}$ feet spans a run of 2 panels, and of 1 panel in the $76\frac{3}{4}$ feet span.

The length and arrangement of the spans was determined by the old structure, the masonry of which was used for the new one. The rebuilding of the bridge, and the building up of the piers to the additional height required, without interruption to the very frequent passage of trains, was an operation at once difficult and interesting, a description of which may be found in a carefully prepared memoir of the work by Mr. Theodore G. Ellis, from which the following memoranda relative to the longest span are taken.

The length of the girder is 173′ 3″; the height between the horizontal plates, 16′ 9″; the width between the centres of girders,

10′ 6″. These girders are divided into 33 panels, of 5 feet 3 inches each, by vertical posts. The end posts are composed of six ⊤ bars, $5'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$, placed in pairs, with two side plates, $25\frac{1}{2}'' \times \frac{3}{8}''$, and one end plate, $12\frac{1}{2}'' \times \frac{3}{8}''$; the whole having a sectional area of 48.18 square inches. The next post has two ⊤ bars, $5'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$, and two side plates, $10'' \times \frac{5}{16}''$, with a sectional area of 14.37 square inches. Posts 3 to 5 are composed of two ⊤ bars, $6'' \times 4'' \times \frac{5}{8}''$, with a sectional area of 11.72 square inches. Posts 6 to 8 have two ⊤ bars, $6'' \times 4'' \times \frac{1}{2}''$, with a sectional area of 9.50 square inches. Posts 9 and 10 have two ⊤ bars, $5'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$, with a sectional area of 8.12 square inches. The posts from 10 to the centre of the truss have two ⊤ bars, of $5'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$, with an area of 7.10 square inches. All the posts have diagonal bracing between the ⊤ bars. They are divided into five spaces, between the chords, by cross plates $5'' \times \frac{7}{16}''$, with diagonals of $2\frac{1}{2}'' \times \frac{1}{4}''$. Between the vertical plates of the chords the ⊤ irons are connected by plates $13'' \times 12'' \times \frac{1}{4}''$.

The posts are placed between, and riveted to, the vertical plates of the chords. Near the ends of the truss there are ten rivets on each side, top and bottom, two being through the angle irons which connect the vertical and horizontal plates of the chord, and the others through the vertical plates. This number is diminished towards the centre according to the sectional area of the post. The rivets through the post and chords are one inch in diameter, and those in the diagonal bracing of the posts $\frac{3}{4}$ of an inch in diameter. The tie bars are in pairs, the first from the end crossing one panel, the second two panels, and the third three panels; being respectively the first ties of the three systems of braces of which the truss is composed. These ties extend a short distance past the centre, there being two beyond the middle panel. This is shown by calculation to be all that is needed for counterbracing, with the heaviest load for which the bridge is intended, or ever likely to be placed upon it.

The dimensions of the ties are as follows, commencing at the end of the girder: —

Two of 8″ by $\frac{5}{8}$″.	Area of pair	= 10.00	square inches.
One " 9″ " $\frac{5}{8}$″.	" "	= 11.25	" "
Two " 8″ " $\frac{5}{8}$″.	" "	= 10.00	" "
Three " 7″ " $\frac{5}{8}$″.	" "	= 8.75	" "
Two " 6″ " $\frac{5}{8}$″.	" "	= 7.50	" "
Three " 5″ " $\frac{5}{8}$″.	" "	= 6.25	" "
Three " 4″ " $\frac{5}{8}$″.	" "	= 5.00	" "
Two " 3″ " $\frac{5}{8}$″.	" "	= 3.75	" "
Three " $2\frac{1}{2}$″ " $\frac{5}{8}$″.	" "	= 3.12	" "

The ties are at an angle of about 45° with the chords, except the first two from the end. The height between the lines of intersection of the ties and verticals is very nearly the same as the length of the three panels each tie crosses. The ties are riveted to the outside of the vertical plates of the chords, part of the rivets also passing through the posts inside of the vertical plates. The number of rivets in each bar is so arranged as to make the sectional area of the rivets fully equal to that of the bar. They are placed in vertical lines, two rows of four each being in the side flanges of the T bar of the post, and the others on lines parallel to these, and diminishing in number, so that the group commences with one rivet. The effective area of the tie bar is thus only diminished by the amount of metal taken out by one rivet hole.

The top and bottom chords are composed of horizontal plates 26 inches wide, extending from end to end of the girders, and varying in thickness from the middle to the ends. At right angles to these are two vertical plates in each chord, placed 15 inches apart, and connected with the horizontal plates by four angle irons in each chord, to which both plates are riveted. The horizontal and vertical plates, except at the ends of the girders, are mostly in lengths of 15′ 9″; the joints coming between the posts of the truss. At the joints in the plates there are covers of the proper size and thickness to make the strength uniform. The rivets in the chords are $3\frac{15}{16}$″ apart, and are one inch in diameter. Between the lower edges of the vertical plates of the upper chords there

are wrought iron distance pieces, one to each panel, by which the two plates are held securely in place, and give the chords additional stiffness.

The top chord at the middle has two horizontal plates, $26'' \times \frac{3}{4}''$, four angle irons, $4'' \times 4'' \times \frac{5}{8}''$, and two vertical plates, $15'' \times \frac{5}{8}''$; making a sectional area of 76.2 square inches. At the end there is one horizontal plate, $26'' \times \frac{5}{8}''$, with angle irons $4'' \times 4'' \times \frac{1}{2}''$, and vertical plates of the same dimensions as the middle; making an area of 50 square inches. The bottom chord is the same as the top, except in the thickness of the horizontal plates, there being at the centre two plates of $26'' \times \frac{5}{8}''$, and $26'' \times \frac{3}{4}''$, and at the ends one plate of $26'' \times \frac{1}{2}''$. The sectional area at the middle is 72.9 square inches, or, deducting rivets, it is 58.4 square inches. Near the ends it is 34.5 square inches.

The horizontal bracing across the top and bottom of the two trusses is formed of T bars placed at right angles to the girders, at intervals of $10\frac{1}{2}$ feet, bringing one over every second post. They vary in size from $6'' \times 4'' \times \frac{1}{2}''$ at the ends, to $4'' \times 4'' \times \frac{3}{8}''$ at the middle of the span. Between these are horizontal diagonal braces of round iron, varying from $1\frac{1}{2}''$ to $1\frac{1}{8}''$ in diameter. The vertical diagonal tie rods are $1\frac{1}{2}''$ diameter at the ends, the rest being $1\frac{1}{8}''$ diameter. These diagonals, both horizontal and vertical, are fitted with nuts and screws for tightening them when necessary.

This span was originally framed with a camber of $3\frac{1}{2}$ inches. It was built with a camber of $3\frac{3}{8}$ inches, measured while the girders were still on blocks and nearly completed. When the supporting blocks were removed the centre of the girders fell $\frac{12}{16}$ of an inch, and after having been passed over by heavy trains they lowered $\frac{9}{16}$ more, making the present camber $2\frac{1}{16}$ inches.

The ends of the girders rest upon cast iron plates; one end is firmly fixed to the pier, and the other is provided with rollers to allow for expansion and contraction. These castings have a packing of oak, boiled in tar, in two thicknesses, of $\frac{3}{4}''$ each, between them and the masonry, and are firmly secured in their places by lewis bolts passing through them into the bedstones.

The superstructure consists of wooden floor beams 7×12, laid across the tops of the girders, 20 inches apart from centre to centre. Upon these rest longitudinal stringers 9×15. The iron work had one coat of boiled linseed oil laid on hot as soon as the work was fitted, and one of red lead before shipment from England, and two coats since completion.

The above span was subjected to a severe test by loading it with railroad bars, in addition to a heavy train of four cars loaded with iron, with the engine and tender; in all, about 220 tons. This would be about $1\frac{1}{4}$ tons to the foot. With this load the deflection of the girders was $\frac{10}{16}''$ on one side, and $\frac{9}{16}''$ on the other. When the load was removed there was a permanent deflection of only $\frac{1}{16}''$ on one side, and none on the other.

Boiler Plate Bridges.

One of the best girders of this class is that erected by Edward S. Philbrick, C. E., at Brighton, upon the Boston and Albany Railroad. Extraordinary care was taken in this work, all of the rivet holes being drilled. The span is 86′ 9″; the extreme height 7′ 6″; the plates are 6′ 3″ wide; the width of flanges 2 feet; the web plates are $\frac{3}{8}$ and $\frac{7}{16}$ thick; the lower flange plates $\frac{1}{2}$, $\frac{5}{8}$, and $\frac{7}{8}$, the angle iron $4 \times 4 \times \frac{1}{2}$. The lower chords are spliced by a single plate 2 feet wide and $\frac{3}{8}$ thick beneath, and two plates, each $7\frac{1}{2} \times \frac{3}{8}$ on top, one each side of the web. The vertical joints are abutted and covered by a batten 8 inches wide on each side, and secured by a double row of rivets. Outside of these battens, on each side, is a vertical angle iron, or stiffening rib, with a base of 3 inches, a projecting flange of 6 inches, and $\frac{3}{8}''$ thick, secured by the same rivets with the battens. Each end of these angle irons is bent out and riveted to the top and bottom flanges. In the middle of each sheet is another vertical stiffener, on each side of the web, of the same dimensions, to check the vibration of the plates; the ends of these being offset and riveted through the angle iron connecting the flange and web. The rivets are

all one inch in diameter. The bridge is for a double track, the middle girder being made stronger than the side ones. The work is very thoroughly braced laterally, and has a heavy flooring of white pine cross ties, 10 × 10, placed 16 inches apart from centre to centre.

Fig. 112 is a transverse section of the general plan of Boiler Plate Bridges, as built by the Portland Company, at Portland, Maine. The proportions given in the table, page 268, are deduced from the general practice of the English engineers, and are entirely reliable.

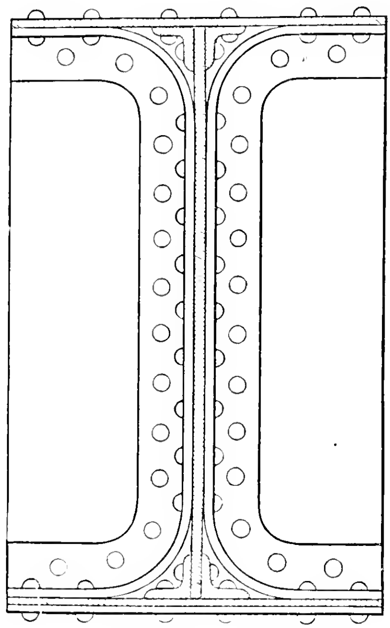


Fig. 112.

Examples of the Portland Company's girders may be seen upon the Grand Trunk Railway, at Presumpscot River, near Portland, at Wild River, in Gilead, and at the crossing of the Connecticut, at North Stratford.

Portland Company's Iron Girder Bridges.

Span in Feet.	Depth in Inches.	Width of each Girder in Inches.	Weight per Lineal Foot of two Girders in Pounds.	Breaking Weight of two Girders. Tons.	Sectional Area of Lower Chord of one Girder, Square Inches.	Distance between Stiffening Plates.	Rivets.	
							Diam.	Pitch.
25	30	16	340	134	9	4′ 0′	$\frac{3}{4}$	$3\frac{1}{2}$
30	33	18	410	137	10	4′ 4″	$\frac{3}{4}$	$3\frac{1}{2}$
35	33	20	430	141	12	4′ 4″	$\frac{3}{4}$	$3\frac{1}{2}$
40	38	21	444	148	$12\frac{1}{2}$	4′ 5″	$\frac{13}{16}$	$3\frac{1}{2}$
45	40	21	484	152	$13\frac{3}{4}$	4 6″	$\frac{13}{16}$	$3\frac{1}{2}$
50	42	21	515	155	$14\frac{3}{4}$	4′ 6″	$\frac{7}{8}$	4
60	48	22	678	162	$16\frac{1}{4}$	5 0	$\frac{7}{8}$	4
70	60	22	730	182	17	5 0′	$\frac{7}{8}$	4
80	72	24	890	210	$18\frac{3}{4}$	5′ 0″	1	4
90	84	26	980	232	20	5′ 0″	1	4
100	94	26	1070	250	$21\frac{1}{4}$	5′ 0″	1	4

The sectional area of the lower chord, as given above, includes the angle iron as well as the chord proper.

The thickness of the plates in different spans is shown below: —

Span 30 feet; web $\frac{1}{4}$ inch throughout.
Top chord at centre $\frac{9}{16}$; at ends $\frac{3}{8}$.
Lower chord at centre $\frac{1}{2}$; at ends $\frac{1}{4}$.
Angle iron $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{7}{16}$.
Stiffening T iron, $5 \times 3 \times \frac{3}{8}$.

Span 50 feet; web $\frac{5}{16}$ inch throughout.
Top chord at centre $\frac{5}{8}$; at ends $\frac{7}{16}$.
Lower chord at centre $\frac{1}{2}$; at ends $\frac{5}{16}$.
Angle iron $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$.
Stiffening iron $5 \times 3 \times \frac{1}{2}$.

Span 80 feet ; web $\frac{3}{8}$ inch throughout.
Top chord at centre $\frac{3}{4}$; at ends $\frac{1}{2}$.
Lower chord at centre $\frac{5}{8}$; at ends $\frac{3}{8}$.
Angle iron $4 \times 4 \times \frac{5}{8}$.
Stiffening T iron $6 \times 4 \times \frac{1}{2}$.

The length of bearing upon the abutments is $1\frac{3}{4}$ feet for the 30 feet girder, $2\frac{1}{2}$ feet for 50 feet, and 4 feet of bearing for the 80 feet span. The cross ties of heavy square timbers rest directly upon the top flange, while a side platform, with a substantial railing, completes the bridge.

CHAPTER XII.

GENERAL REMARKS ON BRIDGE-BUILDING.

In designing iron bridges, we may employ wrought iron throughout, or we may use wrought iron for tension and cast iron for compression. Some builders give the preference to one method, and some to the other. Cast iron is to be preferred on the ground of economy, wrought iron for absolute freedom from hidden defect or internal strains induced by processes of manufacture. The failure of certain large cast iron flanged girders in England, where the material was subjected to tensile and transverse strains, is no reason whatever for not applying the material to the top chords and posts of open-work trusses. Many of the best bridge-builders in the United States employ cast iron for compression members. No bridge has ever failed from a defect in the cast iron top chords or posts. We should avoid, however, reducing too much the size of columns made of this material, on account of the sudden shocks and vibrations to which they are subjected in bridges, or of subjecting the top chords in any way to transverse strains, as by placing the floor upon them in an undergrade bridge. If we take the compressive strength of wrought iron as half that of cast, and the price of the former as twice that of the latter, the relative expense of cast and of wrought iron will be as one to four; besides which is to be considered the greater ease of putting the former material into the various shapes required in bridge work. The remarks of Mr. Fairbairn upon the various defects to which castings are liable, which have frequently been quoted against that material, refer exclusively to cast iron flanged girders intended to resist both transverse and tensile

strains, and not at all to the simple forms used for the compression members of truss bridges. All hollow columns should, of course, be tested by drilling or by callipers to insure a uniform thickness; they should be symmetrically designed to insure freedom from strains in cooling; the metal should be of a uniform quality, and care taken to cool the castings gradually and equally throughout. With these various precautions cast iron will be found a safe, exceedingly convenient, and economical material for the compression members of a bridge. The fact, however, that the ultimate resistance of cast iron to compression is double that of wrought, does not prove that it is superior for this purpose, as the actual shortening or compression under any practicable load is twice as much for cast as for wrought iron. In a simple mechanical point of view, wrought iron is in every way superior; it is in the matter of economy, of first cost alone, that cast iron excels.

While it is universally conceded that iron is ultimately the cheapest, because it is the best material for bridges, there are many railroad companies who do not feel themselves in condition to incur the considerable first outlay necessary, and who, therefore, are obliged to use wood.

In an ordinary wooden bridge, the lower chords are the most cumbersome, weighty, and expensive parts; cumbersome and weighty, because so much surplus material must be introduced to make up the loss from joints, and expensive, because of the extra quantity and length of the timber, and of the elaborate and perfect framing required. They are also the first to decay. Every packing and every joint forms a receptacle for moisture, and decay proceeds rapidly. Every one who has had much experience with wooden bridges knows that the upper chords and braces always largely outlast the lower chords. They are the first to fail, and the most difficult to protect. Their repair or renewal involves the necessity of temporary supports under the whole bridge.

An entire wooden bridge should be housed, to be protected from the weather. The braces would stand exposure compara-

tively well, but the chords are composed, as above mentioned, of several pieces carefully framed together, and so not only retain moisture, but are also very difficult to ventilate thoroughly. It is for their protection, mainly, that covering is required. The upper chords might be covered separately, at a comparatively small expense, but usually, to efficiently protect the lower chords, the whole structure must be housed. This housing is expensive, adds weight useless and injurious to the bridge, is unsightly, adds to the danger from fire, and infinitely increases that from tornadoes — and all for the simple purpose of protecting the lower chords.

By substituting wrought iron for the timber lower chords, leaving the braces and the upper chords of wood, the latter can be covered separately, and thus the only timber parts of the truss exposed to the weather are the braces, which, being single plain sticks, readily shed water, and have every side open to a perfect ventilation. The necessity for housing, with all its serious disadvantages, is entirely avoided, and what was before the weakest and most perishable part of the bridge, is converted into the strongest and most durable. By a simple arrangement in framing, the upper chords and braces may be so constructed that they can at any time be removed without trestling the bridge, or interfering with its use.

The extra expense of the iron over the wood, for which it is substituted, is about equal to the fair cost of housing the bridge, and so the saving in this item will nearly or quite offset the increased cost in the other.

Particular care should be paid to making the connections in timber work so as not to damage the materials. Lateral rods should not pass through the chords so as to act on the grain of the timber; and in all cases the suspension rods should act against the braces by means of a rigid cast iron connection as shown in several plates, and not upon the grain of the timber, as in the old wooden bridges.

In cutting screws upon rods the rod should first be enlarged,

either by upsetting, by drawing it out and folding it back upon itself, by splitting and inserting a wedge, or by welding on a piece, so that the sectional area at the bottom of the screw thread may be at least equal to that of the body of the bar. Many bridge rods, without being thus enlarged, have been weakened from 25 to 35 per cent. by cutting the screw. All screws should bear upon iron, and not upon wood, as they may thus be brought to an equal tension by sound.

Forms of Compression Members.

With regard to the form of compression members, a combination of the several shapes rolled at the iron mills will answer almost every requirement. We give in Plate XXIII. several illustrations. Figs. 1 to 9 are the simple shapes from the rolling mills. Figs. 10 to 29 are combinations of the simple forms, for the chords of trusses. Figs. 30 to 36 are applicable to posts. Fig. 37 is the simple Phœnix column. Fig. 38 is the same with filling pieces riveted between the flanges. Fig. 39 is the compression member referred to on p. 290 in the Lyon Brook Viaduct. Fig. 40 is the cast iron upper chord of the Newark Dyke Bridge, upon a branch of the River Trent, near Newark, England, upon the Great Northern Railroad. These pipes, the length of panel being $18\frac{1}{2}$ feet, have a diameter of $13\frac{1}{2}$ inches, and a metal thickness of $1\frac{1}{2}$ inches at the abutments, increasing to 18 inches diameter, and $2\frac{5}{8}$ metal, at centre of span; the ends of the pipes being accurately turned and fitted, and connected by bolts and nuts. Fig. 41 is the standard section of top chord, circular within and octagonal without, squared at the ends for the connection over the posts, used by Fink, by Bollman, and by the Detroit Bridge Company. Fig. 42 is a Phœnix column in three sections. Fig. 43 is the Keystone column. Fig. 44 shows a Phœnix column, with a T iron riveted into the top for a bearing. Fig. 45 is a composite wrought iron column, by Frederick H. Smith, of the Baltimore Bridge Works. Fig. 46 is a centre section, and Fig. 47 an end sec-

tion of the upper chord of the bridge for carrying the Great Indian Railroad across the River Jumna, at Allahabad, built at the Canada Works, Birkenhead, opposite Liverpool, England. Fig. 48 is the top chord, and Fig. 49 the lower chord of the fine bridge by John Hawkshaw, designed to carry the London Bridge and Charing Cross Railway across the Thames, at Hungerford. The rivet holes in this work were drilled by a machine made for the purpose; all of the holes in one plate (80 in number) being drilled at once through a $\frac{5}{8}$ plate in fifteen minutes. Fig. 50 is a centre section of the cross girders in the same bridge. Fig. 51 is the upper chord used by the Keystone Bridge Company. Fig. 52 is a section of the upper member, or arch, of the bowstring girder of 187 feet span across the Thames, at Windsor, and Fig. 54 the arch of a bowstring girder of 165 feet span across the Shannon. Fig. 56 is the upper chord of a fine bridge over the Ebro, upon the Pamplona and Saragossa Railway, in Spain, the structure being a double triangular, or lattice, in 21 spans, of about 100 feet each. Fig. 55 is the top and bottom chord of the 177 feet span of the Connecticut River Bridge, at Warehouse Point, upon the Hartford and New Haven Railway. Fig. 57 is the upper chord of the fine highway bridge across the Schuylkill, at Fairmount, by Linville.

The sketches above referred to will suggest other combinations, which it is needless to multiply farther. The open forms should be preferred to the closed or box forms, on account of the facility this offers for painting and examination. For the posts in trusses, cast iron hollow cylinders, or posts of cruciform section, enlarged at the ends for connecting, as in the Newark Dyke Bridge, or the forms above referred to, may be employed. For small spans, T irons, riveted together back to back with the tie passing between them, are much used; for larger ones, angle irons may be connected, as in the Canestota Bridge, by a cross lattice.

In the bridge recently completed at Hannibal, across the Mississippi, by the Detroit Bridge and Iron Company, the posts were made of two rolled bars, varying from channel beams 6″

deep, weighing 10 pounds per foot, to I beams 15″ deep, weighing $66\frac{2}{3}$ pounds per foot, connected by a stiff lattice, thus forming an open post rectangular in section, varying from 10 to 24″ wide. These posts reach 6″ below the main coupling pin passing through their foot, the pin seat in the post being re-enforced by a cast iron plate, or washer, riveted to the web of the beams. Columns so made are open to inspection, and accessible to the paint-brush.

Tension members are simple in form, being subject only to a direct pull.

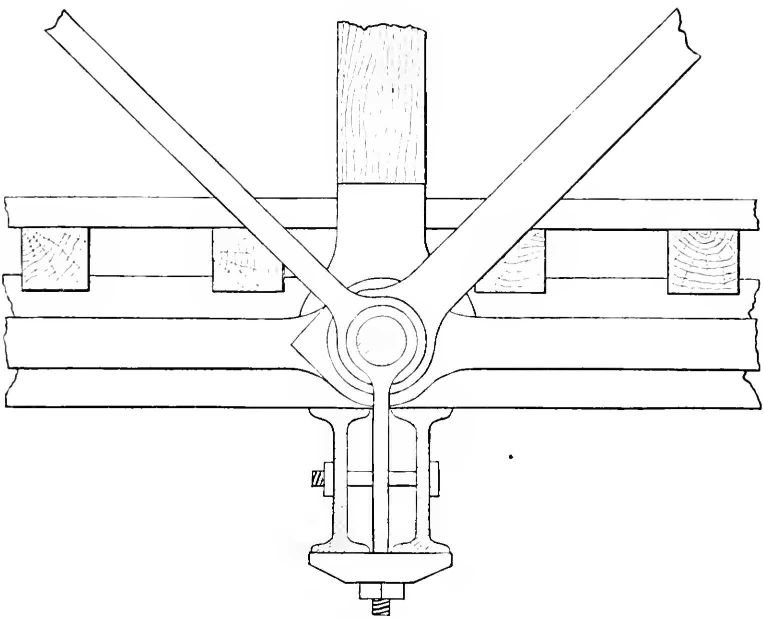


Fig. 113.

The floor of a railway bridge may be supported directly upon the chords, by cross beams reaching from side to side of the bridge, when the chords are of such size and material as not to be damaged by the transverse strain thus produced. A preferable method, however, is that shown in Fig. 113, and also in sev-

eral of the plates where strong cross girders are attached to the foot of each post, and longitudinal girders supporting the track placed thereon. Mr. Humber, in his large work upon bridges (pp. 90–91), compares the two systems for carrying the track, and assuming in the first case the cross beams as 12 feet long, 1 foot deep, and 3 feet apart, and in the second case taking the girders as 10 feet apart, he concludes, for a 90 feet span, that the first case will need $11\frac{1}{4}$ tons, and the second case $9\frac{1}{3}$ tons of iron ; the result thus being in favor of the distant cross girders with longitudinal beams. Practical considerations, however, incline him to prefer the larger number of cross beams. He regards, in his comparison, the chords as being like those most in use in England — very stiff plate work, and well fitted to bear the beams. With the forms generally used in American trusses the case is different, and the second plan will be preferable.

A practice sometimes adopted in this country is to lay a double line of rails across bridges, which is equivalent to applying a continuous guard rail throughout. A heavy substantial timbering is frequently applied, which should be so fixed and guarded that even a derailment will do no damage.

Regard must be paid to the expansion and contraction of materials in designing and in erecting iron bridges. Small rods become heated sooner than large ones, and thus expand quicker. The lower chords of a bridge, on account of their form, generally expand before the top ones. So, too, the counter rods in a truss will often, upon a hot day, expand so as to leave the bridge without a distributing system ; giving to the structure an apparent flexibility, which upon a cool day it does not have. It is thus necessary to make the adjustment of these rods at a mean temperature as near as may be. In putting up a large bridge, it will often be found that the rods are unequally expanded, so as not to go on to the pins until time is given to allow the various bars to acquire an equal temperature. Several methods of accommodating the expansion at one end of a span are shown in the Plates. A combined rocker and roller, used at the Jumna Bridge,

is represented in Fig. 114. Where several spans are continuous, they should, of course, be disconnected, being *fixed* at one end

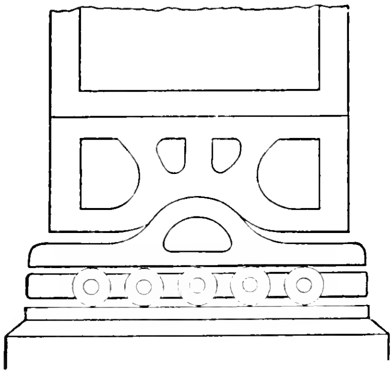


Fig. 114.

only. Fig. 115 shows the connection between the lateral bracing and the chords adopted by the Keystone Bridge Company. The lateral bracing is a very essential feature in all work of this class, and especial care should be paid to it in trestle work and in pile bridging, particularly when the work is upon a curve.

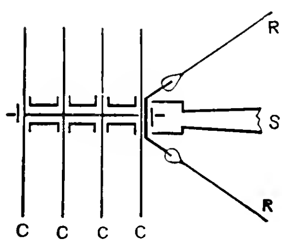


Fig. 115.

When bridges are roofed or covered, the weight of the snow which may lie upon them is a matter to be regarded. Snow weighs from ten to thirty pounds per cubic foot, according to its moisture, and thus adds a considerable amount to the load to be supported.

The effect of wind, also, must be regarded, as it exerts a severe pressure upon a bridge where the sides are boarded in. This is the case only with wooden bridges. The velocity and consequent pressure of wind at different stages is as follows: —

State of Wind.	Velocity in Miles per Hour.	Pressure in lbs. per sq. Foot.
Pleasant gale, ...	10	$\frac{1}{2}$
Very brisk gale, . . .	20	2
Very high wind, . . .	40	8
Hurricane, ...	80	31
Violent tornado, . . .	100	50

There is little danger of a bridge sliding on the masonry, from the force of the wind, if precautions are taken to fasten the work; and when the truss is very high and narrow, and is covered, its safety will be secured by tying it to the abutments and piers by rods running from the top chord to the bridge seat, at as large an angle as possible. Where the roadway is upon the upper chord, such ties may pass from the top chords to the opposite side of the masonry; but when the track is on the lower chord, if the bridge seat is not long enough to give a good angle, the tie may be passed over the outer end of a horizontal strut, let into the masonry.

Safety, durability, and true economy in bridge construction are to be attained by simplicity and uniformity in the design, by the repetition, as far as possible, of similar parts, by the concentration of the material along the lines of strain, by making the strength of the several members proportional to their nearness to their work, thus producing uniformity of strain upon all parts, by the least possible exposure of the surface of iron to corrosion, and of wooden-built beams to the weather, and, finally, by allowing an ample margin for safety, due regard being paid to the proportion of the live to the dead load, as noticed in a former chapter.

Most Economical Length for Spans.

In determining the most economical length for the spans in a long bridge, several matters are to be considered. The total cost of the superstructure and of the piers should, of course, be a minimum. The cost of the abutments, track, and floor may be neglected, being common to all systems. This leaves only the main girders, or trusses, and the piers with their foundations, to be considered. Assuming the cost of a pier to be nearly the same to support a long as a short span, the cost of piers will be as their number, or, inversely, as the length of spans; and the least cost of both piers and bridge will occur when the cost of a pier is just equal to that of one span of superstructure.* The cost of a truss bridge, or girder, depends upon its weight, which again depends upon the length and depth. The cost of a pier is roughly dependent upon its bulk, as far as the masonry goes, but, of course, is quite indeterminate with regard to its foundations. In crossing large rivers, where heavy masses of ice may be brought against the piers, or where the water-way must be reduced as little as possible, a few large and strong piers will be preferable to numerous small and weak ones. On the other hand, in crossing wide and shallow streams, bays, marshes, or dry depressions, where foundations may be had without much expense, and where floods, ice, or navigation need not be regarded, a large number of short openings, with numerous piers, or trestles, is to be preferred. The spans of the Victoria Bridge across the St. Lawrence, at Montreal, which is probably subject to a greater strain from ice than any bridge in the world, have a length from 242 to 247 feet, except the central opening, which, on account of navigation, is 330 feet. In the Quincy Bridge the spans are from 157 to 250 feet. At the crossing of the Quincy Bay, a shallow lagoon, the spans are from 82 to 95 feet. The spans of the

* Whipple, p. 118.

Kansas City Bridge across the Missouri, vary from 177 to 250 feet. The spans of the Louisville Bridge vary from 149 to 245 feet, except the channel spans of 370 and 400. In ordinary cases, for large rivers, with piers of first class masonry from 20 to 40 feet above the water, with common pile or caisson foundations, we may put the spans at from 150 to 250 feet as the most economical, being larger as the foundations are more expensive. The spans employed for the crossings of wide river bottoms, marshes, and bays range from 25 to 50 feet. When trestles are employed in the place of piers of masonry the spans are generally quite short, even when the trestles are very high, as shown in Plate XXVI. Elaborate theoretical discussions upon the above point are of little account, as the peculiarities of each location exercise an important influence.

Ratio between Length and Depth of a Truss.

With regard to the proper ratio between the length and depth of a truss, the best information may be obtained from the practice of American engineers, in which the proportion is seen to vary from $\frac{1}{6}$th, in small spans, to $\frac{1}{10}$th in larger ones. The figures below represent a fair average of this practice: —

Span.	Height.	Ratio.
100	17	$\frac{1}{6}$
150	21	$\frac{1}{7}$
200	25	$\frac{1}{8}$
250	28	$\frac{1}{9}$
300	30	$\frac{1}{10}$
400	40	$\frac{1}{10}$

Several examples, from actual structures, are tabulated below : —

Names of Bridges.	Span.	Rise.	Engineer.
Louisville, ...	400	46	Albert Fink.
Louisville, ...	370	46	" "
Louisville, ...	242	30	" "
Steubenville, ...	319	28	J. H. Linville.
Green River, ...	206	$23\frac{1}{2}$	Albert Fink.
Tygart Valley, ...	200	23	" "
Quincy, ...	250	26	T. C. Clarke.
Quincy, ...	197	24	" "
Quincy, ...	154	22	" "
Kansas City, ...	248	$31\frac{1}{4}$ — 22	O. Chanute.
Kansas City, ...	198	26 — 22	" "
Kansas City, ...	176	22	" "
Kansas City, ...	130	22	" "
Canestota, ...	125	$19\frac{1}{4}$	Chas. Hilton.
Laughery Creek (Highway)	300	30	Frederick Smith.
Connecticut, ...	177	$16\frac{3}{4}$	James Laurie.
Connecticut, ...	140	$12\frac{1}{4}$	" "
Connecticut, ...	$88\frac{1}{2}$	11	" "
Connecticut, ...	$77\frac{1}{2}$	7	" "
Augusta and Albany, .	177	27	T. C. Clarke.
Bollman, ...	96	$18\frac{1}{2}$	Wendel Bollman.
Detroit Bridge Co., . .	125	20	Willard S. Pope.
Ore Hill, ...	130	21	Herthel Hawkins & Burrall.
Burlington, ...	250	26	Detroit Bridge and Iron Co.
Burlington, ...	200	24	" " " " "
Burlington, ...	175	22	" " " " "

One of the most marked and most valuable features of American Iron Bridges was drawn directly from the preceding timber practice, viz., a great depth of girder. The English practice, drawn from the earlier boiler plate bridges, has been continually trammelled by the proportions at first assumed by Stephenson, which, though doubtless correct for solid plate webs, are entirely incorrect for open work girders.*

* In Mr. Hodges' work upon the Victoria Bridge there is published a long letter from Mr. Robert Stephenson to the Directors of the Grand Trunk Railway, defending his adoption of the tubular system for that work, in which he remarks that all bridges must have the same amount of material in the chords, the span and depth being the same, and thus that the question of comparative merit is reduced to the question of the best method of connecting the two chords; and he refers for comparison to several notoriously shallow open work girders in England, and ignores altogether the invaluable element of great depth so widely recognized in the United States. He compares the Victoria Bridge, span 242 feet, and ratio of depth to span 13, weighing 275 tons, with the Newark Dyke open girder, span 240½ feet, and ratio only 15, weighing 292 tons, and claims a balance of 17 tons in favor of the Victoria Bridge. Had the above distinguished engineer compared the Victoria Bridge with such a work as that across the Mississippi at Quincy, of which the span is 247 feet, and the rise 26, thus making the ratio 9½, the weight of which is only 175 tons, he would have presented the two systems in their proper light; but he would have shown that the superstructure of the Victoria Bridge was an enormous engineering blunder.

CHAPTER XIII.

DRAW-BRIDGES, TRESTLES, CENTRES, AND FALSEWORK.

A PASSAGE may be opened through a bridge for the purposes of navigation by several methods. The section to be moved may be lifted vertically, as in many highway bridges, though in the case of a railway girder it must be lifted in one piece and not in two leaves, on account of having a solid bearing when closed. Such a bridge may be counterbalanced by a series of weights attached to the drum by which the lifting chain is wound up, these weights being so arranged as to ground at the bottom of the well one by one as the bridge rises, thus maintaining approximately a balance at all positions of the draw. The same form of girder may be opened horizontally by producing it back over the pier, as in Fig. 8, Plate XXIII., and counterweighting the short end. If such a counterweight be made movable for a few feet, by shifting its position the girder may be lifted from the farther abutment previous to swinging it. When this form is not produced beyond the turning point, it will be necessary to sustain the bridge from above, when open, by long rods passing over a high tower and anchored on the opposite side, the connection at the top being such as to allow the rod to turn as the bridge swings open. In this case the bridge may be lifted by the lever shown in Fig. 1, and opened by the pinion A, Fig. 2, fixed to the pier, working in a curved rack, B, fixed to the bridge. In Fig. 9, a section of the bridge A B is moved diagonally in the direction of the line A C, the part A B being placed upon a carriage for the purpose. In Fig. 10, the section A B is first moved laterally, when the section C D is run directly backwards, thus opening the draw.

The method, however, which is preferable, especially for large draws, is that now very generally in use — the pivot bridge, shown in Figs. 3, 4, 5, and 6, Fig. 4 being upon the Philadelphia, Wilmington, and Baltimore Road, Fig. 5 the draw for the Bay Bridge, at Quincy, Ill, 190 feet long in all, and Fig. 6 the draw in the new bridge across the Hudson, at Albany, 274 feet in all.

Fig. 1, Plate XXIV., shows the general plan of the Keystone Bridge Company's wrought iron pivot bridge, now in use for lengths from 180 to 370 feet. Fig. 2 is a part plan, showing the lower chord, floor, lateral bracing, and lock. Fig. 3 is a top chord plan ; Fig. 4 an enlarged section through the pivot (A on Fig. 1) ;, and Fig. 5 a section through the turning gear and rollers (B on Fig. 1). Fig. 6 is a plan of the upper part of Fig. 4 ; and Fig. 7 a plan of the lower part of Fig. 5. Fig. 8 shows detail of the lock and bearing wedge (at D, Fig. 1) ; and Fig. 9 gives a detail of the friction rollers, the pivot being upon the patent of William Sellers & Co., of Philadelphia. Figs. 10, 11, and 12 show the relative size of the pivot spans of the Kansas City, the Quincy, and the Cumberland River Bridges. When the draw is open, it rests almost entirely upon the central pivot ; the circle of wheels being employed only to balance and guide the draw in its movement. The weight upon the latter may be varied by means of the rods A, B, Figs. 4 and 5. The latches and the wedges are all moved by a lever at the centre of the bridge. A draw of 370 feet total length, and weighing 300 tons, is easily opened in two minutes by four men, with hand levers attached to the pinions on the drum, though a small steam engine is commonly used. Instead of wedges, as in Fig. 8, a cam is sometimes employed to make a solid bearing at the end of the bridge ; and when it is desired to lift the ends of the bridge with a yet greater power, hydraulic jacks, placed within the hollow end posts, worked by pumps, driven by the engines at the centre of the span, have been suggested.

Strains upon Draw-bridges.

To obtain the strains in the pivot draw, it is to be considered in its two positions — open and shut. When open, it may be regarded as two half spans, or cantilevers, connected over the centre, and supporting a uniformly distributed load, viz., its own weight. In this case the strains upon the top chord are tensile, and those upon the bottom are compressive, accumulating from the outer end to the pivot. The strains upon the ties or braces increase in the same direction. Thus, let Fig. 116 represent the half draw open, the length being 100 feet, in 8 panels, of $12\frac{1}{2}$ feet

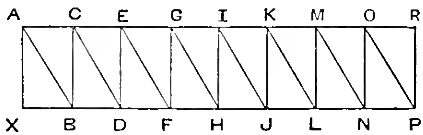


Fig. 116.

each, and the height 20 feet. Take the weight of the bridge as 1600 pounds per foot, or 800 pounds per foot of each truss, or 10,000 pounds per panel. The strains are then as below.

Weight at Foot of the Tie, Pounds.			Tension on the Tie, W × 1.18.	Tension on Top Chord, W × .625.		Accumulated Tension on the Upper Chord.		Accumulated Compression on Lower Chord.	
O P	$\frac{1}{2}$ P, or	5.000	5.900	M O	3.125	M O	3.125	N P	3.125
M N	$1\frac{1}{2}$ P, or	15.000	17.700	K M	9.375	K M	12.500	L N	12.500
K L	$2\frac{1}{2}$ P, or	25.000	29.500	I K	15.625	I K	28.125	J L	28.125
I J	$3\frac{1}{2}$ P, or	35.000	41.300	G I	21.875	G I	50.000	H J	50.000
G H	$4\frac{1}{2}$ P, or	45.000	53.100	E G	28.125	E G	78.125	F H	78.125
E F	$5\frac{1}{2}$ P, or	55.000	64.900	C E	34.375	C E	112.500	D F	112.500
C D	$6\frac{1}{2}$ P, or	65.000	76.700	A C	40.625	A C	153.125	B D	153.125
A B	$7\frac{1}{2}$ P, or	75.000	88.500	A	46.875	A	200.000	X B	200.000

When the draw is shut, it has been common to regard the bridge as two separate spans, the strains being reckoned as in ordinary

cases. The maximum strains from the two positions are then taken for proportioning the parts, due regard being paid to members subject to both compressive and tensile forces. The above mode, in regard to the closed draw, supposes the ends to be so far raised by wedges, cams, or other means, that no tensile strains can come upon the top chord when the bridge is closed. This, however, is not generally correct. The lifting power of a wedge, or cam, as usually applied, is very little, as may be seen by the fact that a heavy train upon one end of the draw of the Kansas City Bridge lifted the unloaded end $\frac{7}{8}''$ from the wedges. The unloaded arm in this condition is under the same strain as when the draw is open, and after the train passes the centre an additional strain is brought upon it until the end is forced down upon the abutment, when it commences to deflect at the centre, and to act as a continuous girder resting upon its supports. The maximum strain over the pivot occurs when both arms are fully loaded; being tension in the top chord, and compression in the lower one. These strains decrease to a point from $\frac{1}{4}$ to $\frac{1}{3}$ of the length of the arm from the centre, where they reverse, becoming compression in the top chord and tension in the lower one. In proportioning the draw, therefore, we may regard each arm when open as loaded, not only with its own weight, but also with the rolling load for the strains upon the one third of the span next the pivot. The greatest compression at the centre of the top chord and tension at the centre of the lower one occurs when one arm is loaded and the other one is empty. The greatest tension at the centre of the upper chord and compression at the centre of the lower one is when the bridge is open. When the draw is shut, and loaded, the central pier carries the weight of each arm and its load for about $\frac{3}{4}$ of its length, and the abutment sustains the remaining $\frac{1}{4}$. The ties and braces should, therefore, be calculated with this point as the centre.*

* The reader is particularly referred upon the above points to pp. 86-87 and 108-114 of the *Kansas City Bridge*. Also, to Chapter X, Vol. I., of Mr. Stoney's work, and to Rankine's Civ. Eng., pp. 287 to 292.

Trestles.

The system of post, girder, and brace work, termed trestling, may be used either for temporary purposes, as where a railway passes over low ground afterwards to be occupied by an embankment, or permanently, over deep and wide depressions where the amount of earthwork or masonry required would be too great. The several figures below show the elements of which trestles are made up. By extending the base, and placing one frame upon another, of course any height may be reached.

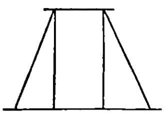


Fig. 117.

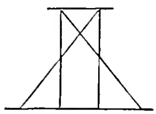


Fig. 118.

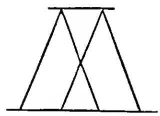


Fig. 119.

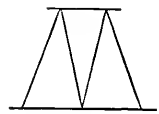


Fig. 120.

The largest example of this method of construction is the great trestle near Portageville, over the Genesee River, upon the Buffalo and New York City Railroad. This immense work is 800 feet long, and 230 feet above the river. It has eight stone piers, 30 feet high, upon which the timber frames are placed; the latter being 190 feet high, 25 feet wide at the top, and 75 at the bottom. Upon the top of all is placed a bridge 14 feet high. To build this structure there were used 1,500,000 feet, board measure, of timber, and 60 tons of bolts. The time occupied in building was 18 months, and the cost was $140,000.

The day for such large structures of wood, however, has passed. It is too perishable a material to be employed where its destruction by fire would so materially affect the traffic of a railway. The standard plan at present is the system of wrought iron post and girder work shown in Plates XXV. and XXVI. Fig. 1, Plate XXV., is an outline of the Lyon Brook Trestle, upon the New York and Oswego Midland Railroad, built in 1869, at the Phœnixville Iron Works, for the Baltimore Bridge Company. This work consists of hollow wrought iron columns, connected by struts and

ties, and supporting at the top trussed girders. There are 24 spans, of 30 feet each, and one span of 100 feet, the whole length being 820 feet. The central trestles are 120 feet high, in four equal stories of 30 feet, the total height above the bed of the stream being 162 feet. Each trestle consists of two legs composed of wrought iron flanged columns, 8 feet apart at top, and sloping at a rate of 1 in 8. The legs of the trestles supporting the short spans have a sectional area of 10 inches in the top story, and an increase of one inch per story downwards; while the bents under the channel span have 20 inches of section in the top story, and the same rate of increase and slope as the common bents. The legs are held apart at the top and bottom, and at intermediate points, by double channel bars of 6 inches section, and tied by diagonal rods of $\frac{3}{4}$ inch section at the common bents; and by wrought iron column struts of 9 inches section, and diagonal tie rods of $1\frac{1}{8}$ inches section at the pier bents. Longitudinally the bents are stiffened by wrought iron column struts of 9 inches section and diagonal tie rods of $1\frac{1}{8}$ inches of section in the pier squares, and by one inch rods and wooden struts in two pieces 4×12 between the pier squares and the masonry, a continuous longitudinal strut of iron being objectionable on account of the expansion of so great a length. All of the ties, struts, and column legs are firmly and squarely connected by cast iron joints, caps, and bases in the bents, and all of the rods are adjustable. The chords of the short trusses are each in two sticks, $7\frac{1}{2} \times 15$, being made of wood, as they act as longitudinal struts between the caps of the bents, and all of the truss chains of both long and short spans connect with each other on pins over each bent, while there is a through system of lateral tie rods of one inch section between the chords from end to end of the bridge, thus completing the perfect continuity of the structure.

The centres of the short spans rest upon wrought iron post frames 8 feet deep, the post having a section of 6 inches, in four angle irons, the top strut being a 6 inch rolled beam, and the bottom strut a 3 inch T iron, and diagonal rods $\frac{3}{4}$ of an inch in diameter. The truss chains have $3\frac{1}{8}$ inches of section, in two bars $2\frac{1}{2} \times \frac{5}{8}$. The

pins are $2\frac{1}{2}$ inches in diameter. The chords of the channel span are composed of a 15 inch rolled beam riveted up as a filling strip between the two halves of a rolled iron flanged column. The beam contains $15\frac{1}{2}$ square inches of section, 12 of which are used to resist the transverse strain, the remaining $3\frac{1}{2}$ being added to the 14 square inches in the column to resist the compression, thus securing the beam as the best form for the transverse strain, and the cylinder as the best form to resist compression, and no consequent loss of metal. The truss is 20 feet deep at the centre and quarters, and 10 feet at the eighths. The top struts are 6 inches in section, and the lower ones 3 inches, all of wrought iron, with diagonal rods $\frac{7}{8}$ of an inch in the main and quarter post frames, and $\frac{3}{4}$ of an inch in the eighths. The main post is a rolled column of 12 inches section supported on 9 inches of section on the main chain, in four bars per truss, $3 \times \frac{3}{4}$ inches. The quarter post is a rolled column of 10 inches section resting on the quarter chains of 4 inches section, in two bars of $2\frac{9}{16} \times \frac{3}{4}$ inches. The eighth post has 6 inches section, in four angle irons, resting on $2\frac{1}{2}$ inches of section on the eighth chain, in two bars, $2 \times \frac{5}{8}$ inches. The cross ties are 8×9 inches, placed two feet apart, with guard stringers 6×8 inches bolted and notched down over them. Every third cross tie extends out, on each side, 9 feet from the centre line, and supports the sidewalks and handrails.

This structure is designed to carry the heaviest machinery and rolling stock at express speeds, using a factor of safety of 5 in the spans, and of 7 in the bents. The contract price was $48,960. Fig. 1, Plate XXV., is an outline elevation. Figs. 2 and 3, transverse sections at the top and bottom of the quarter posts of the 100 feet span. Figs. 6 and 7, side elevations of the top and bottom of the eighth post. Figs. 8, 9, and 10, side elevations of the top, middle, and foot of the channel columns. Fig. 11, the centre, and 12, the end of side of small trusses. Fig. 13, connection of small posts, wooden struts, and diagonals of small trestles. Fig. 14, foot of small columns. Figs. 15 and 16, adjustable lug for holding the sag of the chain at its crossing of the quarter post.

The later works of this class avoid the use of the longitudinal wooden struts above mentioned, the objection to so long a line of iron being overcome by treating each three trestles, or pair of bays, as a separate pier, connected together without any provision for expansion ; while between the piers thus formed the longitudinal struts are omitted, and sliding joints made in the chords, or the longitudinal girders, so that the expansion does not accumulate. This arrangement is shown in Plate XXV. ; which is a portion of the Rapallo Viaduct, built by the Phœnixville Bridge Company for the New Haven, Middletown, and Willimantic Railway. This work has 46 spans of 30 feet, and is from 30 to 60 feet in height.

The most remarkable example of trestle-work yet constructed is the Verrugas Viaduct, upon the Lima and Oroya Railway, in Peru, built in 1872 by the Baltimore Bridge Company. This immense work, shown in Plate XXVI., crosses a deep gorge in the Cordilleras, about 50 miles from Lima, and upwards of 5000 feet above the sea. It consists of three Fink trusses of 100 feet each, and one of 125 feet, and three piers 50 feet long and 15 feet wide on top, making a total length of 572 feet from face to face of abutment. The piers are 145, 252, and 178 feet in height. The whole work is of wrought iron, except the joint connections, which are of cast iron. Each pier is composed of 12 Phœnix columns — 8 of six segments, and 4 of four segments, arranged as in the Plate. The outside columns have a batter of 1 in 12. The piers are divided into tiers of 25 feet each in height, united by strong cast iron joints, to which are attached the longitudinal and transverse struts, and the transverse and horizontal rods for bracing. Each of the six-segment columns contains 20 inches of sectional area, and when the structure is fully loaded, it is strained to 4612 pounds per inch.

The spans of the above work were made short, in accordance with the original plan, which was to frame them in the bed of the ravine and lift them bodily into place. The piers were to be erected from below, and within themselves, by means of an inside balanced crane lifted from tier to tier. The method employed,

however, was different. Two wire cables were stretched over temporary wooden towers, at each end of the viaduct, and properly anchored. On these cables travellers were placed, by which the material for the piers was taken from the end of the track, at the abutment, and lowered into place. The 125 feet span was erected on scaffolding, and a strong temporary truss was framed, and moved, by means of the cable, to the several openings in succession, from which the spans were swung into place.

Pier No. 3, 178 feet high, was erected in 18 days; No. 2, 252 feet high, in 12 days; and No. 1, 145 feet high, in 12 days. Span No. 3 was swung in 22 hours; No. 2 in 16½ hours; and No. 1 in 18 hours. The whole time consumed, including all preparations, was 3½ months; and the time actually occupied in raising, 55 working days.

The Baltimore Bridge Company constructed also the Running Water Viaduct, upon the Nashville and Chattanooga Railway, 692 feet long, and 115 feet high at the deepest part; the Sidney Center Viaduct, upon the New York and Oswego Midland Railway, 1500 feet long, and 100 feet high at the deepest part; Clarke's River Viaduct, upon the Elizabethtown and Paducah Railway, 460 feet in length; the trestle-work at St. Charles Bridge, 4318 feet long; the Arequipa Viaduct, upon the Arequipa and Puno Railway (South America), 1500 feet in length, and 55 feet high at the deepest part, making with other works of the same kind a total length of nearly two miles.

The height at which a trestle becomes more economical than an embankment depends much upon the local conditions affecting the price of earth-work, and ranges from 25 to 50 feet. When the time for construction is limited, the trestle is almost always preferable; and the deeper the depression, the greater its advantage. A certain relation exists, also, between the height and the distance between the trestles. A structure of this kind placed upon heavy screw piles furnishes a good mode of crossing wide bottoms, where stone-work is expensive, and foundations difficult to obtain. It is also suited for highway bridges passing over railways, where it may be employed almost without masonry.

CENTRES.

The centres for arches from 5 to 15 feet span may be made of planks, as in Fig. 121. For longer openings the arch may be

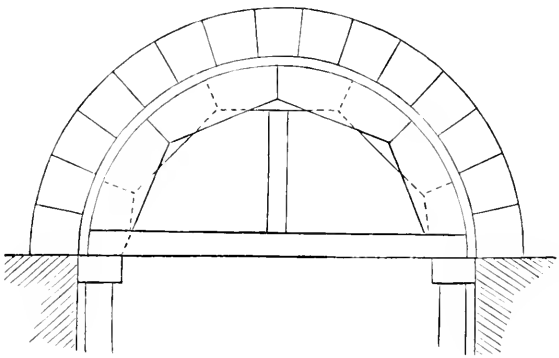


Fig. 121.

stiffened by radial pieces, while the centre of the horizontal tie may be supported from the ground beneath, when the location admits of such an arrangement. When there is no support except the abutments, the system of bracing, shown in Figs. 122 and

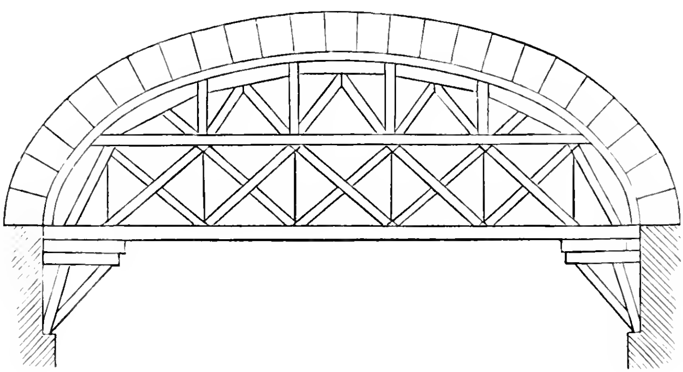


Fig. 122.

123, may be employed. The centres used at the Bletchingly Tunnel, on the railway from London to Dover, England, are shown in

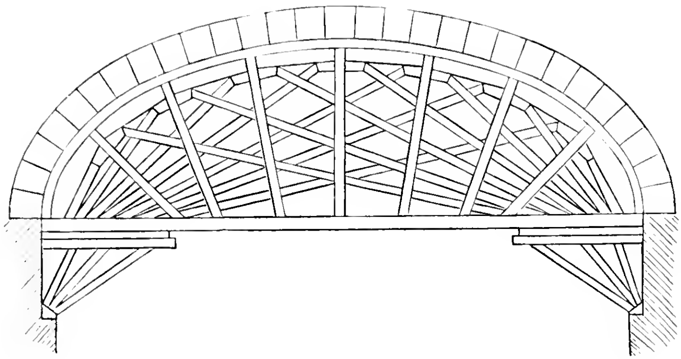


Fig. 123.

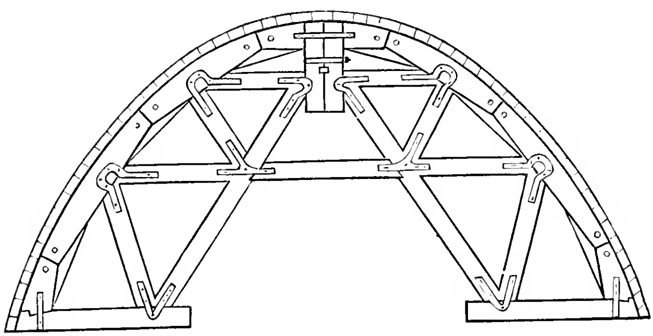


Fig. 124.

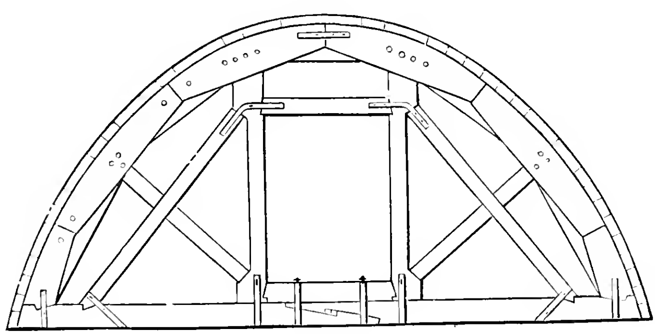


Fig. 125.

Figs. 124 and 125; the first being the leading centre, left open at the middle to allow the passage of the inclined braces reaching back from the heading, and the second the following centre.

The centres for large arches require to be removed very gradually, for which purpose they are supported upon the arrangement shown in Fig. 126. A and B are termed striking plates, A

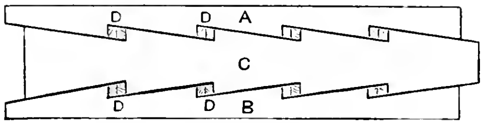


Fig. 126.

being the lower member of the centre, and B the upper member of the support. The long notched wedge C is held in place by the wedges D D, which, being loosened, allow the wedge C to be driven backwards, thus letting down the upper plate A, and, of course, the centre resting upon it. An ingenious mode of lowering a heavy centre, which has been employed in Europe, is shown in Fig. 127, where A is the upper striking plate, B the lower one,

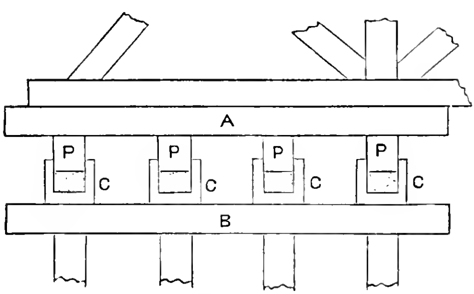


Fig. 127.

C C iron cylinders partly filled with sand, upon the upper surface of which rest the plungers P P. To strike the centre, small holes in the lower part of the cylinders, stopped by corks, are opened,

when the sand runs out, and allows the upper plate A to settle gradually.

Centres are strained in a different manner as the construction of the arch progresses. A load upon the haunches causes a settling at those points, and a corresponding rising at the crown. By loading the crown, as the work goes on, this movement may

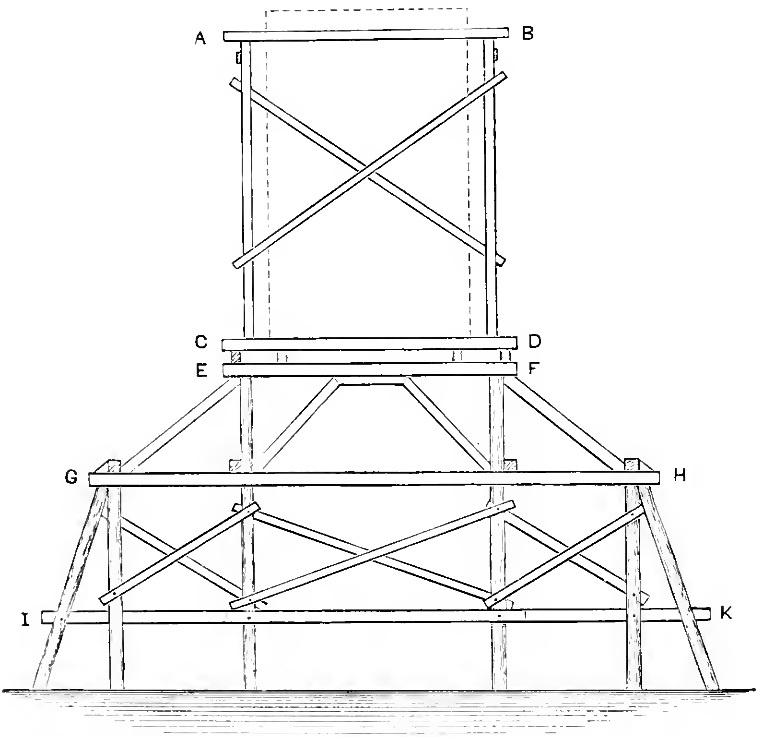


Fig. 128.

be, to a great extent, checked. The work should be carried up uniformly, upon both sides of the crown, at once, otherwise the distortion of the centre will be unsymmetrical.

The ribbed frames above shown are placed from 1 to 5 feet apart, according to the load to be supported. Upon the top of

the centres is placed a course of thick narrow planks, or small timbers, termed a lagging, upon which the arch stones are placed.

The Falsework employed in the raising of bridges into place, is merely a rough but rigid piece of trestling, supported either upon the ground directly, or upon piles ; generally, the latter, when the structure is over water. Fig. 128 represents the falsework employed at the Quincy Bridge ; the dotted line representing the transverse outline of the truss. The lower, or pile trestles, G H I K, are placed about 20 feet apart, longitudinally, while the upper trestles, A B C D E F, are 13 feet apart, or the same distance as the length of the panels of the bridge.

CHAPTER XIV.

FOUNDATIONS.

FOUNDATIONS may be placed upon solid rock, upon firm and incompressible earths, or upon soft and yielding materials, and each of these cases may occur upon the land or beneath the water. Solid rock, stony, compact soils, and hard clays, which are practically incompressible, and do not yield laterally, furnish at once a good foundation, care being taken to dress off the upper surface of rock enough to remove decayed and loose material, and to obtain the necessary form for the superstructure; and, in the case of earth, to excavate to a sufficient depth to be beyond the reach of frost, and to avoid liability to being uncovered and exposed. Pure, clean gravel and dry, sharp sand are incompressible, so long as they are not allowed to spread out laterally, or are not acted upon by water, and thus, with proper caution, will furnish a sure foundation. In compressible and yielding materials, such as common clay, loam, wet and spongy soils, especially when the bottom yields unequally at different points, care is required to prevent a dangerous settling of the masonry above. In such cases a trench filled with sand will often be found an effective and economical mode of distributing the load, care being taken to confine it laterally. Another method consists in making an open raft of two courses of round or square timber, crossing at right angles, and filling the spaces with concrete. A platform of thick plank may be placed upon the raft, or the masonry may be placed directly upon the lower timbers and the concrete. When the weight of a heavy structure is thrown upon a few small points of support, they may be made the piers and abutments of a

series of inverted arches, by which the whole surface is made to assist in bearing the load. The side walls of tunnels are frequently arranged in this manner. Wet, marshy, and yielding soils may be compacted by driving a large number of short piles as close together as they can be driven without interfering with each other, commencing at the outside row. Around and over the tops of the piles thus driven concrete may be packed, the whole mass of earth, piles, and concrete making thus a firm bed for the masonry. Piles used as above need not be over from 6 to 9 inches in diameter, and from 6 to 12 feet long.

Pile Work.

In ordinary practice piles are employed, either to penetrate a yielding material, in order to secure a bearing upon a solid substratum, or to support a superincumbent load by the friction of their sides against the material through which they pass. According to Mr. McAlpine, piles of the same size, driven into sand by the same weight of ram, and the same fall, but to different depths, have sustaining powers, as the square of their exterior frictional surfaces; and a pile driven home in fine sand with a ram of 1 ton, falling 30 feet, will sustain a ton for each superficial foot of its friction surface. This, however, exceeds the support due to its sectional area. The same engineer gives the following general formula for the extreme sustaining power of piles driven into sand, one third of which may safely be used in practice, when there is no danger from vibration of the structure being communicated to the piles, or from scouring action of the water:—

$$P = \dot{8}0\ (W + .228\sqrt{F} - 1).$$

Where P is the extreme supporting power of the pile, and W the weight of the ram, both in tons, and F the fall in feet.

The rule given by Major John Sanders, of the United States Engineers, is as follows:—

$$W = \frac{R \times h \div d}{8}.$$

In which W is the weight in pounds that may safely be placed upon the pile; R the weight of the ram in pounds; h the fall of the ram, and d the penetration of the pile, both in feet.

Professors Rankine and Mahan both give in their Manuals 1000 pounds per square inch of area of the head as the safe load on piles, driven until they reach firm ground; and $\frac{1}{5}$ as much, or 200 pounds per inch, when they resist by friction against the material through which they pass. Perronet regarded 112,000 pounds as not too great for a 12 inch pile, and allowed 25 tons for one of 9 inches in diameter. Rondelet gives the working load upon timber piles surrounded by earth as from 427 to 498 pounds per inch. Stoney remarks, that as far as the strength of the timber is concerned, we might safely load piles surrounded by the ground with $\frac{1}{5}$th of the crushing weight of wet timber, which is equivalent to $\frac{1}{10}$th the crushing weight of dry timber. A common specification is, that piles shall not be depressed more than a $\frac{1}{4}$th of an inch at the last blow of a 2500 pounds ram falling 30 feet. Professors Rankine and Mahan give the French standard of resistance, viz., a penetration of not more than $\frac{1}{5}$th of an inch from 30 blows of an 800 pounds ram falling 5 feet. According to the above, a pile 12″ diameter, and 20 feet long, and thus having an area of head of 113 inches, and a friction surface of 62.8 feet, if driven home by a 2000 pounds ram falling 25 feet, would sustain as a safe load in practice, by McAlpine's rule, 60,800 pounds, and would have an ultimate frictional resistance in sand of 62.8 tons. Sanders's rule gives for the same 312,500 pounds. Mahan's and Rankine's rule for bearing gives 113,000 lbs., or for friction 22,600 pounds. By Perronet, the safe load would be 112,000 pounds; by Rondelet, about 450 × 113, or 50,850 pounds; and, finally, $\frac{1}{10}$th of the crushing resistance of wet hard wood would be 400 × 113, or 45,200 pounds. The piles in the foundation of the High Level Bridge, Newcastle, England, were 40 feet long, and driven through sand and gravel to rock. At the Royal Border Bridge, over the Tweed, the piles were driven from 30 to 40 feet into gravel and sand. The maximum pressure that can come upon the above,

supposing none of the weight to be carried by the intervening concrete and planking, would be 70 T., or if the piles were 15″ diameter 57 T. per square foot, or 887 pounds per inch.*

The following general deductions are given by Mr. McAlpine, from extended and careful experiments made by him during the construction of the United States Dry Dock, at Brooklyn, N. Y.†

"The sustaining power of a pile driven home, the resistance which it meets with, and the force of the blow, are, of course, equal. The subsequent settlement of the material around the pile, however, increases its supporting power, and this varies in different kinds of soil. This increased support cannot, however, be relied upon if there is any vibration in the piles, or if there is a scour about them.

"In foundations under water there will be a degree of fluidity given to the material by the operation of driving, which lessens the frictional resistance to the penetration of the pile, but the superior gravity of sand to that of water allows it to settle in close contact with the pile, and gives a greater coefficient of support, than if it was driven through the same kind of material in a dry state.

"There is no increased force of blow obtained by a fall of more than 40 feet, as the friction on the ways increases so rapidly that no increased velocity is attained by falling from a greater height. Thirty feet is the most useful fall, as the machines are commonly made.

"The steam hammer of Mr. Nasmyth has been very successfully applied to the driving of wooden piles. In this engine, a great number of very rapid blows from a very heavy ram produces an excellent result. The pile is not allowed to recover from one blow before it receives another.

* See also paper No. 5, upon Practical Engineering, by officers of the U. S. Engineers, Lieutenant Colonel J. L. Mason, on the Resistance of Piles.

† The valuable paper of Mr. McAlpine may be seen in full in the Franklin Journal, vol. 55, 1868, pp. 98 and 170. From this paper the deductions above are extracted.

"The force of a blow from a Nasmyth hammer is much less, but the effect is much greater than with the common machine. A pile of 35 feet was driven home in 7 minutes by the Nasmyth hammer, while an hour or more was required to drive a similar pile with the ordinary engine. The blows of the Nasmyth ram were given at intervals of less than a second of time, and before the material displaced by the vibrations of the preceding blow had time to subside, and, therefore, nearly the whole force of the blow was employed in the displacement beneath the pile. In the other machines the blows were given at intervals of a minute, by which time the vibrations had ceased, and the material had partially settled around the pile, so that a considerable part of the force of the blow was consumed in overcoming the friction along the sides, and in the removal of the sand by new vibrations, leaving only a small part of the force to displace the earth at the bottom. This effect would be greater in loose and partly fluid material than in dry or compact soil."

Gauge piles are those driven to mark the outline of any work. Guard piles are those employed either temporarily or permanently to protect structures from ice, drift, etc., in rivers. Sheet piles are squared timbers driven close together, and matched, or tongued and grooved, in order to form the side of a coffer dam, or like work intended to exclude the water. Piles can be driven as near to each other as $2\frac{1}{2}$ feet from centre to centre, but large piles driven very close will force each other up. In such cases the piles should be driven butt down, commencing at the centre of the area and working towards the outside. For ordinary cases, 3 feet apart from centre to centre is near enough. Piles may be driven in a vertical or an inclined position, according to the angle at which the ram slides; though, of course, this must not be far from the vertical when the ram falls by gravity. Spring piles, used for bracing pile work, are driven vertically, and drawn over by a tackle or by screws and bolted to the upright piles, thus forming a very stiff brace. When piles are to be driven very deep they are often in more than one piece, care being taken

to make a secure and true butt joint at the connection. When the pile is required to be driven only a short distance beyond the reach of the ram, a short post, called a follower, is placed on top of it to transmit the blows.

Piles driven into very hard material require to be shod with iron to preserve the point, and the top is protected from the blows of the ram by being bound with an iron ring, which is removed when the pile is driven. Oak or elm is the best material for piles. The piles being driven, the tops are sawed off level, and a heavy open work floor of square timber placed upon them, as in Fig. 129. Any soft material about the heads of the piles is then removed, and concrete filled in to the level of the top of the platform, upon which the masonry may be commenced, either with or without a course of planking ; or the floor may be omitted altogether, and a bed of concrete filled around and over the piles, upon which the lower courses of masonry may be placed, as in Fig. 130.

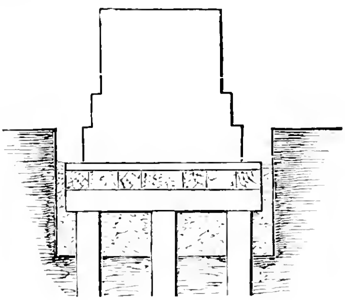


Fig. 129.

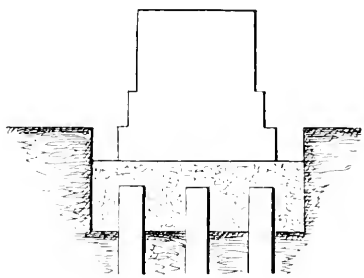


Fig. 130.

When a mass of clay is underlaid by a soft material it is often best to rely upon a strong platform and a bed of concrete, and not to cut up the firm upper stratum by piling. Where the upper bed is of a loose and yielding substance with a hard bed beneath, into which the piles are driven, especial care will be required to prevent lateral motion in the work. When not too deep the surface material may be replaced by broken stone, thrown in so as to make a permanent bed around the piles.

The general plan of the foundations at the Quincy Bridge, shown in Figs. 131, 132, consisted of a platform placed upon concrete and piles, and the whole surrounded by a crib, rip-rapped

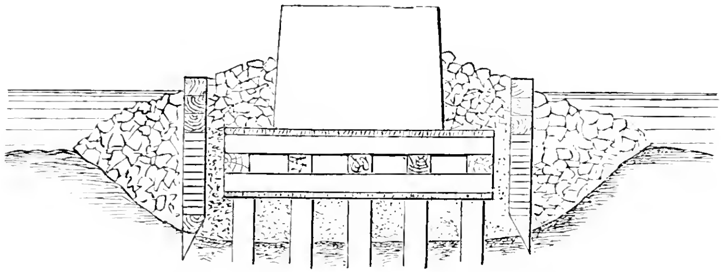


Fig. 131.

on the outer sides. The Mississippi, at Quincy, has a gentle slope, a wide area, and does not scour, as it does below the entrance of the Missouri, and thus the above method is applicable.

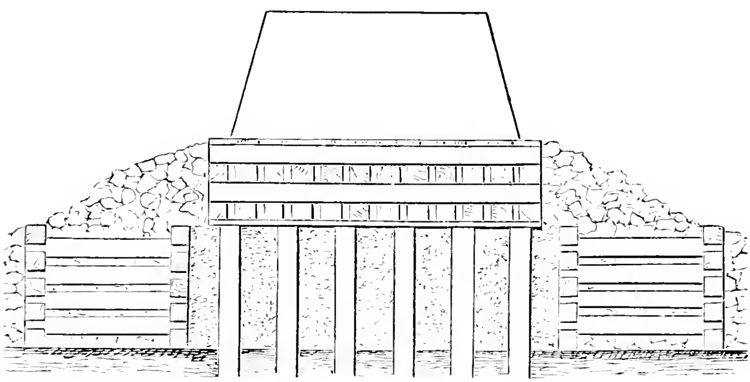


Fig 132.

The crib or caissons, formed of solid elm timbers a foot square, securely bolted together, were sunk 9 feet below low-water mark, or from 10 to 12 feet into the sand where deepest. Into this box white oak piles, 3 feet from centre to centre, were put, and sawed

off $5\frac{1}{2}$ feet below low water. The bottom was then levelled up to within 2 feet of the heads of the piles with stone, and the upper 2 feet filled with hydraulic lime concrete, or beton, carefully levelled. Upon this a grillage made of 3 courses of pine 12×12, placed 6 inches apart, and bolted together and planked, was floated into position, or built in place, and suspended by 10 screws, $2\frac{3}{4}$ inches in diameter, with 4 threads per inch. These had an upset head, and passed through a cast iron washer at the lower end, which suspended two $1\frac{1}{2}$ inch round rods, passing through the platform, and secured by nuts on top of the cast iron washers, which were turned by the diver after the platform was lowered, releasing the large screws, and leaving the rods in place. The nuts of the large screws were turned by four men, with a 6 feet lever, the greatest depth lowered being 15 feet, the average 8 feet, and the time $7\frac{1}{2}$ hours of 40 men. The cost of lowering was from \$50 to \$60; the immersed weight being about 112 tons.

THE SCREW PILE.

The Screw Pile, invented by Mr. Alexander Mitchell, consists of a post, either of cast iron, wrought iron, or timber with a cast iron foot, having a socket at the top, a point at the bottom, and a large spiral flange. The outside diameter of the screw is generally much larger than the body of the pile, with a pitch from $\frac{1}{4}$ to $\frac{1}{3}$ of the diameter. It is screwed into the ground from above, and furnishes a large bearing surface upon the bottom. These piles may be worked into the soil without much disturbing it, and may be easily unscrewed if necessary. They have been successfully applied to every kind of bottom except hard rock, penetrating even into chalk, and have been driven 12 feet into the coral of the Florida reefs. Such piles have been employed both for the foundation and the superstructure of bridge piers, and are particularly applicable to the lower story of the large iron trestles now much in use, especially where stone is difficult to obtain for piers, or in crossing wide depressions of marshy land.

Disk Piles.

In founding a viaduct for carrying the Ulverstone and Lancaster Railway across the River Leven, at Morecambe Bay, England, Mr. Brunlees employed hollow cast iron piles 10 inches in diameter, spread out into a broad disk at the bottom. Experiments shew the sustaining power of the sand to be about 5 tons per square foot, and as each pile was to carry 20 tons, the diameter of the disk was fixed at 2′ 6″, giving an area of 4.90 square feet, or 24.50 tons of sustaining power. In sinking these piles, a hole in the bottom received a wrought iron pipe 2 inches in diameter, which, passing upwards, was connected by a flexible hose with a pump which forced water down, and blew the sand out from under the disk, until the pile sank from 7 to 9 feet. To settle the sand when the sinking was done the pile was driven 2 inches by a heavy ram.

Pneumatic Tubes.

From the driving of piles, properly so called, we pass to the sinking by other methods of large hollow columns of iron, which have been applied both as the foundations, and as foundations and superstructure combined, for the piers of bridges in difficult locations. The atmospheric system of Dr. Potts consists in exhausting the air from a hollow cast iron cylinder, when the outside pressure forces the earth immediately beneath the pile to its interior. At the same time the pile sinks into the opening thus made, both by its weight and by the atmospheric pressure from above. The earth is removed from the interior of the pile, and, when sunk to the required depth, the column is filled with concrete or masonry. This process, however, is especially adapted to sandy bottoms, but does not answer in hard material, or when the piles in their descent encounter large rocks, logs, or like obstructions. In such cases the process has been changed from the vacuum to the plenum, i. e., instead of exhausting the air

from the column, a body of air is forced in, and condensed to an amount sufficient to drive out the water, when workmen descend into the tube and excavate the material, passing it up through the tube itself, which descends under a proper control until a suitable foundation is reached, when the column is filled with concrete or masonry. The following description of the columns sunk for the Harlem Bridge, at New York, by William J. McAlpine, illustrates fully the process.

The Harlem Bridge.

The pile consists of a number of hollow cylinders, A A, Fig. 133, 6 feet in diameter, $1\frac{1}{4}$ inches thickness of metal, and 9 feet long,

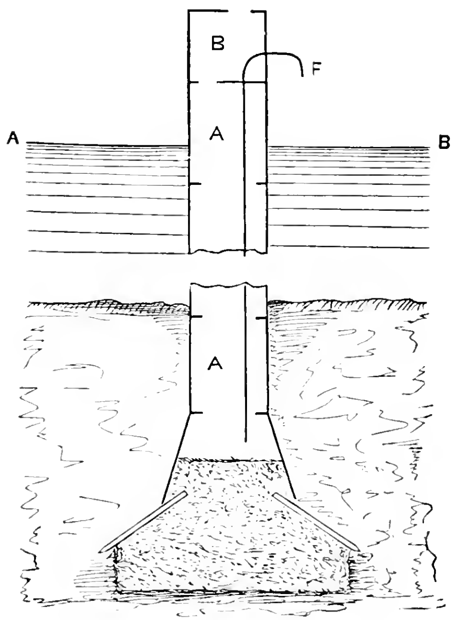


Fig. 133.

provided with flanges on the inside, by which they are bolted together until the required length is obtained, the lower edge of the

lower one being chamfered to about ¼ inch thick. Another cylinder, B, usually of boiler iron, called the air-lock, is placed on top of the column, the sides being of the same diameter as the columns, with a top and bottom plate of cast iron, in which are man-holes, closed air tight by plates opening below. The top and bottom of the air-lock have small cocks by which a connection may be made between A and B, and between B and the open air. Leading from the outside of the air-lock, near its bottom, are two curved tubes, F, 4 inches in diameter, which pass through the bottom, and are closed by cocks. An air pump connected with one of the curved pipes condenses the air in A, and as this pressure increases it forces the water out through the open bottom. This continues until the pressure of the air equals that due to the head of water outside, and the water has all been forced out. The workmen then enter the air-lock, and closing the upper man-hole, open the cock in the bottom admitting the compressed air from below. When the pressure is equalized the lower man-hole plate falls, and the workmen pass down to the bottom to excavate the material which is raised to the air-lock. When the column has been entirely cleared, care is taken to see that no logs, boulders, etc., remain under the lower edge of the column, and the men ascend to the air-lock, shut the lower valve, and open the cock in the upper plate, allowing the compressed air to escape. When the pressure in the air-lock is equalized with that outside, the upper man-hole cover falls, the men can pass out, and the material be removed. Finally the large pipe, F, is opened, and the compressed air is allowed to escape quickly, and thus the upward pressure within the column is removed, allowing the weight of the mass to act with an effect similar to that of a blow, while at the same time the rapid inrush of water at the bottom scours the material out from under the lower edge, thus removing the resistance to sinking the column. The friction of the outside of the column against the material through which it passes is greatly diminished by the current of water passing down its surface on its way to the inside. If no rocks, trees, or other like

obstructions are encountered, the column will continue to settle quite rapidly while the air is escaping, and afterwards until the material has stopped scouring under the edges, and has become compacted hard enough to bear the weight. The settling in one operation will frequently amount to 10 or 12 feet, or even more. When obstructions are met, the column is recharged with air, and the workmen descend and remove them, after which the sinking goes on as before. The greatest depth beneath the water at the Harlem Bridge was about 50 feet, the pressure due to which was about 22 pounds above the atmosphere, or 37 pounds in all; though this was frequently, on special occasions, increased by an atmosphere.

Mr. McAlpine refers particularly to the extension of the concrete at the base of the column, represented in Fig. 133, and adopted at the Harlem Bridge. A column of 3 feet diameter, with an expanded iron base of 6 feet, and a further expansion of the concrete to 10 feet, and driven 40 feet into the earth, would have an external frictional support of about 180 tons, and a support from the bottom area of as much as 800 tons; and this increased support by expanding the base would be obtained by an expenditure of less than $100. The depth of the concrete must, of course, be proportioned to its width. At the bridge above referred to, wooden sheet piles, 5 feet long, 3 inches wide, and $1\frac{1}{4}$ inches thick, were driven at an angle of 30 degrees, as in Fig. 133, in sections of a few feet only in width at a time, and quickly excavated and filled. The time occupied in driving the columns an average depth of 25 feet below the bed of the river in sand, through boulders, logs, etc., and finishing the concrete base, was from 7 to 20 days for each column. When the earth at the bottom was so compact as to refuse a passage to the water from within the column, it was forced up through a siphon pipe within the pile, and discharged through the curved neck at the bottom of the air-lock. Mr. McAlpine states that a pier made of 2 columns filled with concrete, 8 feet diameter, braced together, and protected by a wooden or iron starling, will resist,

without the slightest injury to itself, the heaviest blows that a steamer, or a drift of timber, or ice can give it; and that such piers for a bridge across the Mississippi may be finished ready for the superstructure in six months, or even less, without regard to weather or freshets.

The Pneumatic Process in Europe.

The plenum process was first introduced by M. Triger, a French engineer, in sinking coal shafts, from 3 to 6 feet in diameter, through wet soils. At the Rochester Bridge, England, 14 columns put down by this method, 7 feet in diameter, support a grillage, on which a masonry pier 70 feet long and 17′ 8″ wide is built. At the Chepstow Bridge the pier is made of 10 columns, 6′ 4″ diameter, 6 of them being carried up to the bridge seat. The Macon Bridge, over the Saone, in France, has piers made of 3 cast iron columns, 10 feet in diameter, and 1½ inches of metal, placed 13 feet from centres, and sunk from 32 to 40 feet into the bottom, or 50 feet below low water. Above this they are surmounted by smaller tubes, 8′ 3″ in diameter, joined to the lower tube by a conical base. The whole is protected by a mass of concrete 19½ feet deep, enclosed in sheet piling. The columns are braced together, filled with concrete, and capped with cut stone. The piers of the Theiss Bridge, in Hungary, at Szegedin, built by M. Cézanne, in 1858, consist of 2 columns each, 10 feet in diameter, placed 13 feet from centre to centre, and sunk 39 feet below low water, or 29 feet into the soil. After the tubes were sunk by the plenum process, wooden piles were driven inside of them to a further depth of 20 feet, to consolidate the bottom, and support the concrete with which they were filled up to low water. These columns are protected by a timber ice breaker, and surrounded by concrete and riprap enclosed in sheet piling. The mode first used for sinking the tubes was by suddenly allowing the compressed air to escape, which had the same effect as striking a blow upon the top. This, however, carried the tubes

down by jerks, and often threw them out of line. To avoid this, at the bridge of Bordeaux, over the Garonne, built in 1859–60, the tubes were forced down by hydraulic pressure; the full compression of air being maintained. These piers are each made of two cast iron columns, each 11′ 9″ in diameter, sunk 52 feet below low water. The castings are in sections 4′ 8″ high, and 1½ inches of metal, with inside flanges, and filled with concrete. The time occupied in sinking the 12 tubes, for 6 piers, was 13 months. The pier of Mr. Brunel's great bridge at Saltash, was founded on a rock 82 feet below high tide, covered with 17 feet of mud. A boiler plate cylinder, 88 feet high and 35 feet diameter at bottom, was employed; and to reduce the working space within the tube the cylinder was divided into 2 parts, the bottom being provided with a spherical dome under which the plenum was to be maintained, and the upper portion so arranged as to be removed after the masonry was built up. The health of the workmen employed at so great a depth, and under a pressure of 2 or 3 atmospheres, was somewhat impaired, which suggests that 80 feet is about the limit for this process. Seven bridges upon the Warsaw and St. Petersburg Railroad have been founded by the plenum process. At the Kowna Bridge, over the Nieman, each pier has 4 columns, 11′ 6″ diameter, sunk 39 feet below the water, being driven 33 feet into a granitic sand, and resting on a bed of hard clay. All parts exposed to shocks are 2½ inches thick. In these tubes the compressed air was confined to a working chamber at the bottom, 15 feet high, connected with an air-lock at the top by two 30 inch surface tubes of wrought iron. By this arrangement the volume of the compressed air required was much reduced, and trouble from leakage in the outer tubes avoided. The weight for forcing down the pile was obtained by admitting water above the working chamber. This work has stood well since 1861, notwithstanding a change of temperature of 135°, and bad jams of ice 2 and 3 feet thick. The cylinders are filled with concrete, made of 1 part Portland cement, 2 parts sand, and 3¼ parts broken stone and brick.

With regard to the duration of cast iron columns in water, Mr. McAlpine states that soft graphitic iron changes its character in very foul water, and slightly so in soft water; but that when the carbon is combined, no such change occurs, and only a slight external oxidation takes place, which thereafter serves as a protection.

The Coffer Dam.

In founding, in water, from 5 to 25 feet deep, a coffer dam will often be found an economical mode of obtaining a good bed. When the water is not over 3 or 4 feet deep, a simple bank of clay, well put down, the bottom being first cleared of loose material, will serve to keep the water out. When the depth is greater, a double row of sheet piling, filled in with puddle of clay, or other material, will be required, the thickness of the dam depending upon the height. The following have been used:—

Height.	Thickness.
5 feet.	6 feet.
10 "	8 "
15 "	10 "
20 "	12½ "
25 "	15 "

The sheet piling for the sides of the dam may be arranged as in Fig. 134, where A A are guide piles, B B top strings or wales, between which are the sheet piles, tongued and grooved. C D and E show three modes of matching the planks to make tight joints. Another method of arranging the sheet piling is shown in Fig. 134 (A), P P being the piles, and W W the outside string. Inside is another string piece to guide the sheet piles, and inside of all another string. The same members are shown in section in Fig. 134 (B), which shows both sides of the dam, with the filling X. When the dam is emptied of water, the sides will have to resist the pressure from without; and if not thick enough to do

this by their own strength, they must be braced. The great difficulty in the coffer dam is to keep the outside water from working under the dam to the inside. This has been checked in some cases by the following method. The outer row of sheet piling is first driven, the loose material within removed, and replaced with a bed of concrete, into which, while still soft, a second row of

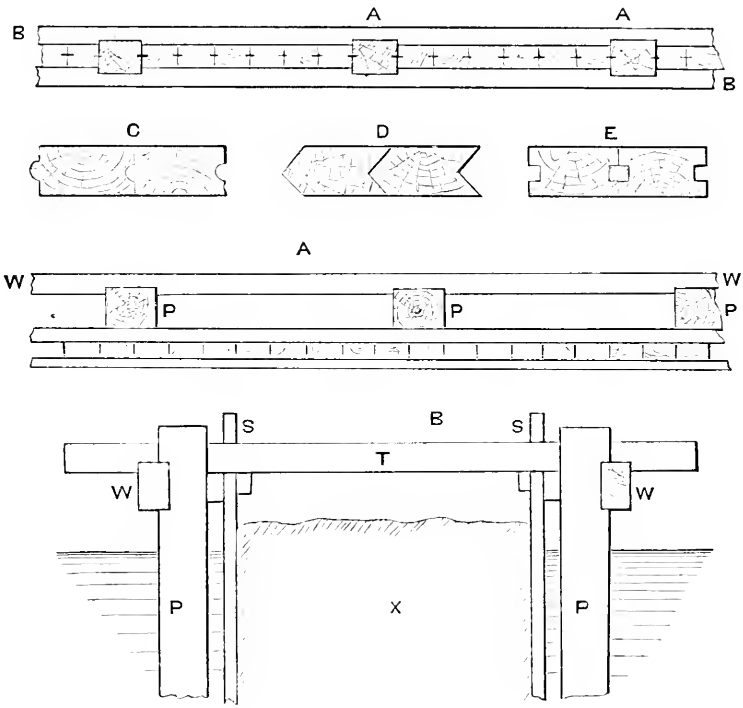


Fig. 134.

piles is driven, and the puddling filled in, and the water pumped out from the interior.

When the bottom is of rock, so that piles cannot be driven, the dam may be built in sections, and floated to the site and sunk, the several parts connected, and the puddling put in. This was done

at the Victoria Bridge, Montreal, where the depth of water at the summer level was from 5 to 15 feet, the bed of the river being of limestone, with large boulders on the surface. Dams, 188 feet long and 90 feet wide, pointed on the up stream end, made with double sides, were floated out and sunk at the proper places, the hinder part being made so as to be removed, thus enabling the dam to be taken away to winter quarters. These dams, when launched, drew only 18 inches of water. The bed of the river being rock, into which piles could not be driven, at distances of

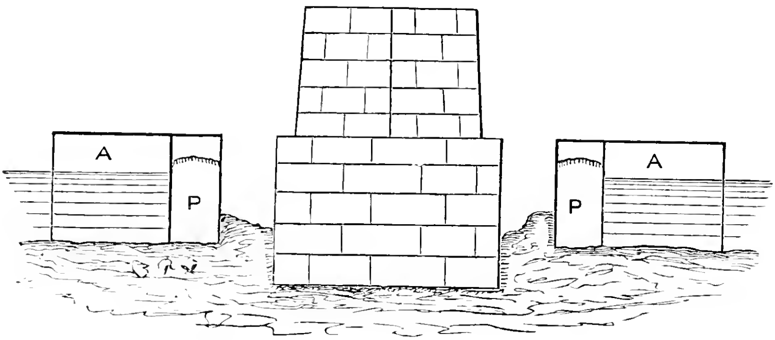


Fig. 135.

20 feet all around the outside strong piles, sliding in grooves, were provided, and through the centre of some 10 or 12 of these piles a 2 inch iron bar, with a tempered steel point, shaped for drilling, was put, which being drilled about 2 feet down, kept the feet of the piles in place. The section of these dams at right angles to their length was as in Fig. 135. A A being the double sides of the dam, and P P the puddle walls inside.

Methods of Puddling.

With regard to the proper mode of making the puddling, the common idea is to use clay alone, or clay and sand. Mr. McAlpine, however, who has had, perhaps, more experience in difficult foundations than any other engineer in this country, in a valuable

paper read before the American Society of Civil Engineers, states that he very much prefers a puddle of gravel to one of clay, and that the first coffer dam at the Brooklyn Dock gave way chiefly because it was filled with clay, but that the one built by himself withstood a great pressure, because it was filled with gravel. "Particles of clay," says Mr. McAlpine, "are cohesive, and a vein of water, never so small, which finds a passage under or through clay, is continually wearing a larger opening. The particles of fine gravel, on the other hand, have no cohesion; a vein of water first washes out from the gravel the fine particles of sand, and the larger particles fall into the space, and these small stones first intercept the coarser sand, and next the particles of loam which are drifted in by the current of water, and thus the whole mass puddles itself better than the engineer could do with his own hands. An embankment of gravel is comparatively safe, and becomes tighter every day. One of clay is much tighter at first, but is always liable to breakage. For the same reason the piling trench should be filled with gravel, so that if any vein of water escapes through or below the sheet piling, the weight of the gravel will crush down and fill up the vein before it can enlarge itself enough to produce danger."

The Caisson.

In deep water the coffer dam becomes very expensive, on account of the size and length of the piling required, and the large amount of bracing needed to resist the inward pressure of the water. In such cases the caisson has often been resorted to; this being simply a box, in which the masonry is built, and afterwards sunk to the proposed site, which is to be properly prepared, either by dredging or by piling. Finally, the sides are removed, the bottom being left underneath as a platform. Guide piles will be needed to insure the proper descent of the caisson, and to control the equilibrium as the mass descends.

THE KANSAS CITY BRIDGE.

The foundations of the railway bridge across the Missouri, at Kansas City, presented a variety of difficulties, which were successfully overcome by the engineer, Mr. Chanute, in a manner at once economical and instructive. The channel piers were carried down through from 2 to 40 feet of sand to the rock, in a rapid current, by means of open bottomed caissons, lowered partly by screws and partly by gravity, the sand being dredged out by machinery. The caisson for pier No. 1 was a timber box, 70×19.5 feet, floated into position, and guided in its descent by eight posts, four on each side, 60 feet long and 16 inches square, with a central hole 3 inches in diameter, from end to end, through which a 2 inch steel drill, welded to an iron rod 65 feet long, might be worked. These posts sank by their own weight through the 2 feet of sand, and were made fast by working the drills 2 feet into the rock. They were then well braced together, being about an inch nearer at the top than the bottom, to secure clearance in lowering the caisson. Cross timbers attached to the posts carried each 2 screws, 20 feet long and 2 inches in diameter. The caisson weighed 72 tons, and thus each screw held 9 tons. It was provided with a false bottom, and admitting water above this ease and regularity in the descent were secured. When the caisson reached the sand the screws were removed, and the box sunk 2 feet lower to the rock, by blowing away the sand with a jet of water, thrown out of a copper nozzle, attached to a 3 inch hose, supplied by a pump above, the nozzle being used by a diver. A tight joint was made by sheet piles surrounding the caisson, driven against the rock, and by packing gunny bags filled with clay around the outside, in a trench, excavated in the sand by a jet in the hands of a diver. These bags were further surrounded by hay covered with a bank of clay, protected from the water by a canvas tarpaulin, the whole covered with a layer of clay and stones. The caisson being braced internally, the bottom cleared, and tested with a drill, the masonry was put down.

At pier No. 3, where the rock was 30 feet below low water, and overlaid by 22 feet of sand, and the water 17 feet deep, the caisson was let down on to the sand by screws S S, Fig. 136, and the sand removed by an endless chain dredge, having a capacity of 50 cubic yards a day. A tight joint was made inside by placing bags

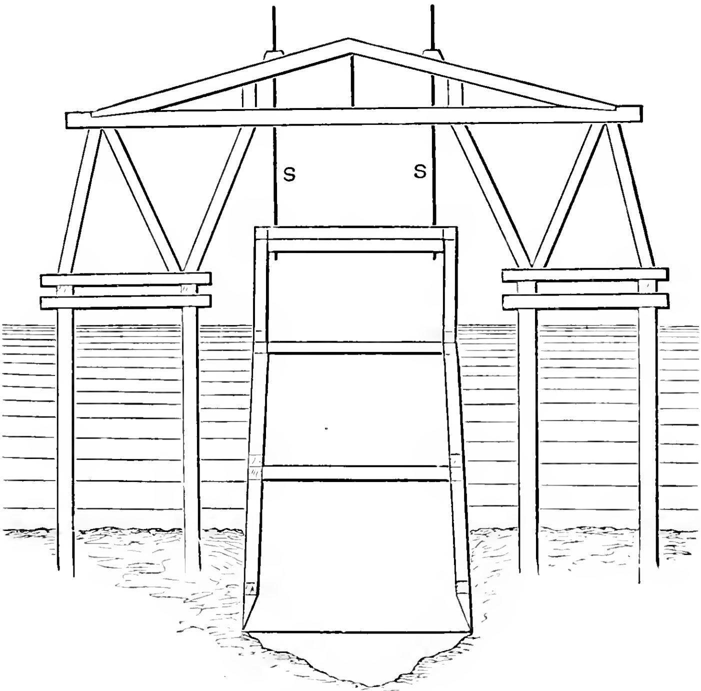


Fig. 136.

of freshly mixed beton around the interior edge, and a foundation of beton, 22 feet deep put in, upon which the pier was built.

At pier No. 4 especial difficulties were encountered. A pier of masonry was to be built in position above water, and sunk through 40 feet of sand to the bed rock, by excavating the sand beneath with dredges working through wells left in the masonry, guiding the mass in its descent by screws, keeping the top of the

masonry above the water. The caisson was to bear the weight of 40 feet of masonry above, and the pressure of the sand and water against the sides. It was built of heavy square timber, with a double course of 3 inch oak plank outside, and with an inside wall, and three cross walls, so arranged as to brace the caisson thoroughly within and to support the masonry above, dividing the interior of the caisson into four bell-shaped chambers, in which the air could, if necessary, be compressed, as in the plenum pneumatic process. To put the caisson down, the spaces between the double side walls were filled with beton, the dredges were mounted, working through rectangular tubes of wood, four in number, built above the several air chambers. When the caisson reached the bottom, the wells and the air chambers were filled with beton, the whole mass reaching up to about 25 feet above the bottom, and upon this the masonry of the pier was commenced.

Among other interesting details in the above work, the use of the guide piles fastened to the rock bottom by steel drills, the employment of water jets to loosen sand about the caisson, the bracing of piles in rapid currents, the sinking of larger piles by a water jet carried through a gas pipe laid in a groove cut in the side of the pile, and discharged through a hole in the point of the iron shoe, may be mentioned as very useful expedients, applicable to a great deal of the foundation work upon railways. The reader is especially referred to the excellent report upon the above work by its engineers for very numerous details for the overcoming of special difficulties.*

The amount of friction between the sides of caissons and the earth through which they are sunk, depends, of course, upon the material. For sand, Mr. Clarke reckons the friction per square foot of rubbed surface, equal to the depth in feet to be sunk, multiplied by 19 pounds, and the product multiplied by a coefficient varying from 0.4 to 0.7.† Mr. Chanute, from his experience at the Kan-

* Kansas City Bridge — Chanute and Morrison.

† Clarke, Quincy Bridge, p. 29.

sas City Bridge, considers the average friction, in pounds, on each superficial foot of caisson in contact with the sand, to be eight times the average depth in feet of the cutting edge below the surface of the sand; this applying, however, only to that particular locality.*

The Diving Bell.

The diving bell is used more for examination beneath the water, and for laying of work when the foundation may be prepared from above, as by piling, than for anything more difficult. It is simply an iron box, generally rectangular, 5 or 6 feet square, and about the same height, weighing from 6 to 8 tons, raised and lowered by means of a windlass. A flexible pipe for supplying it with air at the proper degree of compression, is connected with the air pump above. It is furnished with thick plates of glass in the top to admit the light, and with signal cords for communication between the workmen in the bell and those above who control its movement.

The Kehl Bridge.

A combination of the pneumatic process, the caisson and diving bell, has been very successfully employed, both in Europe and in America, for the putting down foundations in deep water. It was first used at the fine bridge built by the governments of France and Baden across the Rhine at Kehl, opposite Strasbourg. At this place the bed of the river is composed of an almost indefinite depth of sand, gravel, and silt, which has been known to scour to a depth of 55 feet below low water, and during freshets attains a velocity of from 9 to 11 miles an hour. The arrangement adopted is sketched roughly in Fig. 137, in which A A A A is a strong boiler plate box, open at the bottom, with

* Kansas City Bridge. p. 99

three holes in the top. Through the central hole a tube was inserted, reaching downward a little below the bottom edge of the caisson, and extended upwards, as the caisson sunk, so as to terminate at the height of the working platform. From the smaller holes ascended the pneumatic tubes, through which the com-

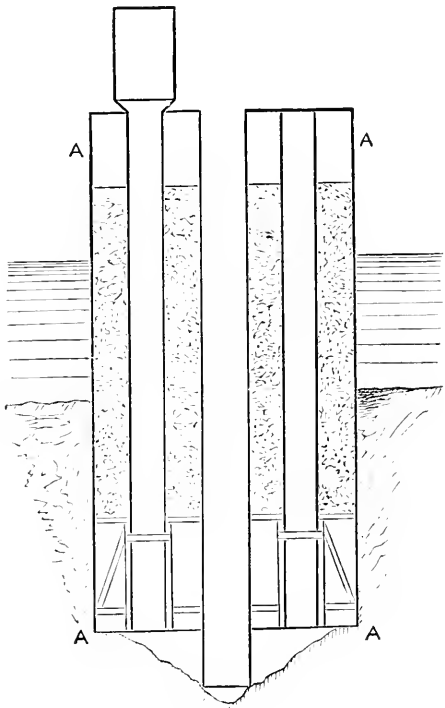


Fig. 137.

pressed air was sent into the caisson, and the workmen ascended and descended. Each air tube was used alternately, the air chamber being fastened to one while the other was being lengthened. The caisson being lowered to the river bed by screws, attached to the platform, and the pier commenced upon the top of the inverted box, or bell, compressed air is forced in through the tubes,

the water driven out, and the workmen commence to feed the earth under the edge of the central tube, through which it is raised to the surface by a vertical dredge, consisting of a series of buckets on an endless chain. These buckets, extending below the bottom of the central tube, made thus a hole into which the material was easily pushed. The pier was built of concrete, faced with hammer-dressed stone, and carried up so as to keep the weight but slightly in excess of that needed to overcome the friction, and within the power of the screws to sustain.

The above method substitutes a single mass of masonry for isolated tubes. It makes the weight of the tubes useful in sinking, hastens the work, and reduces the expense. In more recent operations the sides of the caisson have been carried up above the working chamber or bell, so that the height of the masonry may be at all times adjusted to the weight required without regard to the water line. At the shore piers of the Kehl Bridge, which had each 4 dredges and 8 pneumatic tubes, the first was put down in 55 working days, the second in 31, to a depth of 65′ 9″ below low water, the greatest depth worked at being 72′ 9″ below the water line, and the greatest pressure employed about 3 atmospheres. In order to have still water to work in, the site of the pier was enclosed in sheet piling, and substantial platforms roofed over were provided, so that the work might go on night and day, in all weathers. The cost of the shore piers, 77 × 23, and put 65′ 9″ below low water, were, for the French side, $141,360, and for the German side, $117,180. The intermediate piers, 57 × 23, and sunk to the same depth, cost from $92,000 to $94,000 each, and this at the low price of labor common in Europe.

The St. Louis Bridge.

One of the most interesting and most difficult works yet undertaken in America, is the magnificent bridge across the Mississippi, at St. Louis. The river at this point is 2200 feet wide at high water, and has a rock bed overlaid by about 15 feet of sand on the

western shore, and by nearly 100 feet on the eastern side, the rock sloping from west to east, the sand being nearly level. The scour of this sand takes place to depths so great that it was not deemed safe to found the piers on anything short of the rock bottom. The bridge is to consist of 3 spans, the centre one being 515 feet, and the side ones 497 feet each, in the clear. The

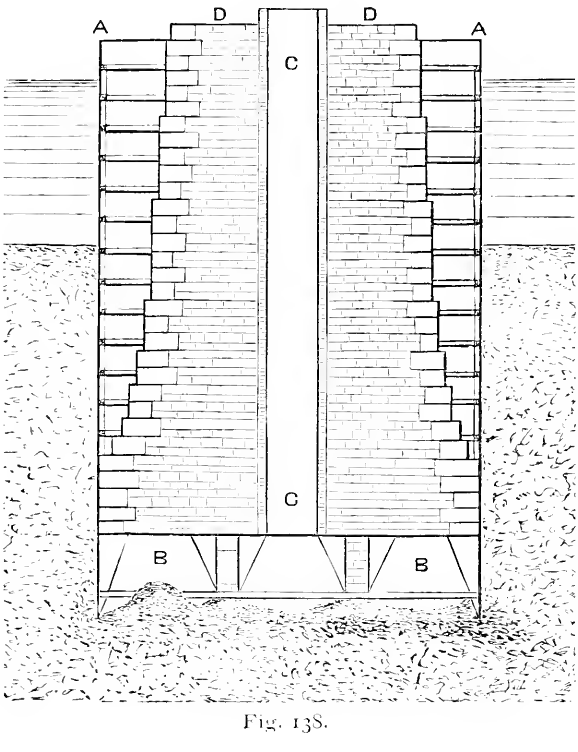


Fig. 138.

height of the eastern pier, when completed, ten feet above low water mark, will be 97 feet, and that of the other 69 feet above the rock. About 78 feet in depth of sand will be encountered in sinking the one, and 50 feet in the other, with about 20 feet of water on the site of each pier. The base of each pier is 82 feet

long — the eastern one being 60 feet wide, and the other 48 feet wide. The larger one covers an area of 4020 square feet, and the other 3360 square feet. At ten feet above low water mark each pier is 78 feet in length, by 38 feet in width, and at that point each one has a sectional area of 2600 square feet. The general plan of the caisson used in sinking the eastern pier is shown in Fig. 138, in which A A is the caisson, B B the air chamber, entered by means of air-locks, C a large well, affording access to the works, and D the masonry of the pier. A brief description of the caissons and of the sinking the masonry, extracted from the reports of the engineer, Captain James B. Eads, will be found in the Appendix.

The use of hollow cast iron columns, and of compressed air caissons, admits of penetration to great depths through all kinds of material, and of getting to solid bottoms, which could otherwise hardly be reached. Columns 6 feet in diameter have been driven 25 feet in as many days, through one continued mass of boulders, and in one case through the hull of a vessel, where the wood was in perfect preservation and strongly bolted, with a delay of but two days. Such columns have been used from 7 inches to 20 feet in diameter, and driven into sand from 7 to 75 feet below low water. By caissons piers have been sunk from 40 to no less than 90 feet, and this at all stages of water in all weather, and during both day and night. The compressed air method becomes expedient at depths beyond 33 feet; at 100 feet the high degree of compression required in the air becomes injurious to the health of workmen of certain constitutions. The process is an expensive one, and only to be employed in those difficult locations where the older methods are inapplicable.

CHAPTER XV.

MASONRY.

Varieties of Stone.

The varieties of stone employed in engineering operations are the granites, limestones, and sandstones. Slates, bricks, and artificial stones, the latter made of broken stone and cement, are also used in many works. Rock is generally found in beds, divided by joints, or seams, at which the natural adhesion is broken, and the layers may be separated. When the quarry shows no natural lines of separation, a fissure is produced by drilling a line of holes at equal distances from each other, into which conical steel pins are driven, thus splitting the stone, the pins being placed in the plane of the required seam. In opening a quarry, the surface rock is commonly found to have been softened for a few feet by atmospheric agencies, the material thus requiring to be removed for a short distance in order to come to the solid rock. When stone cannot be obtained, brick forms, in many cases, an excellent substitute, being made from clay earths, which are found in almost every locality. They are well fitted for good work, and are easily transported and handled. Stone is employed almost entirely to resist a crushing strain, as in the voussoirs of an arch, or in the courses of a pier. The ultimate resistance of the several varieties of stone and brick to compression is as follows: —

Slates, ...	10,000 to 20,000	pounds	per	inch.
Limestone, ...	7,000 to 15,000	"	"	"
Granite, ...	6,000 to 12,000	"	"	"
Marble, ...	3,000 to 9,000	"	"	"

Sandstone,	2,000 to	8,000	pounds	per	inch.
Fire Brick,	1,500 to	1,700	"	"	"
Strong Red Brick,	800 to	1,200	"	"	"
Pale Red Brick,	500 to	800	"	"	"
Brick laid in cement,		500	"	"	"

The factor of safety for stone should not be less than 10. The strain upon the columns supporting the domes of St. Peter's at Rome, St. Paul's in London, and other large European churches, ranges from 300 to 600 pounds per inch, or from $\frac{1}{7}$th to $\frac{1}{16}$th of the crushing weight.

The expansion of stone by heat, per degree of Fahrenheit, has been determined as below by Mr. Adie, of Edinburgh, and by Professor Bartlett, in this country.

Variety of Stone.	Bartlett.	Adie.
Granite,	.000004825	.000004722
Marble,	.000005668	.000004861
Sandstone,	.000009532	.000005833
Best Stock Brick,	....	.000003000

Limes, Mortars, and Cements.

Nothing is more important in the construction of masonry than good mortars and cements. Under this head are to be considered limes, cements, sands, common mortar, hydraulic mortar, and concrete, or beton.

Lime is obtained by burning off the carbonic acid from pure limestones, when it is put up in air-tight barrels, and is the unslaked lime of commerce.

Rich, fat, or common limes contain less than 10 per cent. of impurities, double or treble their volume in slaking, are soluble in fresh waters, and when made into a paste harden in the open

air, but do not harden under water or in damp situations, or when excluded from the air.

Poor limes contain silica, alumina, iron, manganese, and other impurities, in all from 10 to 35 per cent. They slake slower than the fat limes, increase less in volume, dissolve less in water, and only harden in the open air. They are seldom used.

The hydraulic limes contain silica, alumina, magnesia, and oxides of iron and manganese. With from 10 to 20 per cent. of these the limes are termed slightly hydraulic, and set under water in from 15 to 20 days. With from 17 to 24 per cent. of impurities the lime is termed hydraulic, and sets in 6 or 8 days; with from 20 to 35 per cent. the lime is termed eminently hydraulic, setting in 1 to 4 days; they all slake slowly with but a small elevation of temperature and a slight increase of volume. Hydraulic cements contain a yet larger amount of the foreign ingredients named above, even as much as 60 per cent. They do not slake at all after calcining, but set very rapidly under water, even in a few minutes. They do not increase their volume by being made into paste, and do not shrink in hardening, and may thus be used without sand.

Sand is the product of the decomposition of the various rocks; the silicious sands from granites, schists, and quartzose rocks being the best and the most common. This material is found either on sea or river shores, or in alluvial excavations. The latter, termed pit sand, is generally rougher, with a sharper grain than that from beaches or rivers. It is, however, often mixed with loam or other earths, which require to be removed by washing before it can be used.

The various mortars are formed by the mixtures of lime, sand, and water. The amount of lime should be a little more than would fill the spaces in the sand, since each grain of sand should be entirely surrounded by the lime or cement. The volume of these spaces may be found by filling a measure with sand, and then pouring in water until the spaces are filled, the volume of water being that of the spaces. The sand performs no chemical

part in mortars, but is employed chiefly to obtain the necessary bulk, being cheaper than lime or cement. It controls the shrinkage of the cementing matter, making it uniform, and prevents cracking. It diminishes the strength of hydraulic cement, though it is generally used for the sake of economy. The coarser sands should be used with the rich limes, mixed sands (coarse and fine) with the weaker hydraulic limes, and fine sand with the eminently hydraulic.

"The object," says Lieutenant Wright, in his excellent treatise on mortars, "which we propose to attain in mingling sands with a cementing material, is to form, as cheaply as possible, compositions which, exposed to all vicissitudes of weather, and even placed under water, may nevertheless become hard and solid, attach themselves strongly to building materials, and attain in the end a resistance superior to all disturbing forces."

For structures on dry land, exposed only to the ordinary atmospheric changes, we employ a mortar of rich lime, sand of the medium and coarser sort, and in some cases a small percentage of cement. For works under water, foundations, and structures exposed to dampness, we employ the hydraulic limes and the hydraulic cements, mixed with the finer varieties of sand.

The richer the lime, the greater the proportion of sand which it will bear; and the more hydraulic the lime, the less the proportion of sand. According to Vicat, the best proportions, by measure, are, —

2.4 sand to 1 pure slaked lime paste.
1.8 sand to 1 hydraulic lime paste.

At Fort Warren, Boston Harbor, the proportions were, —

For stone masonry, —

1 cask (240 pounds net) lime, or 8 cubic feet stiff paste.
2 casks (650 pounds net) hyd. cem., $7\frac{1}{2}$ " " " "
$38\frac{1}{2}$ cubic feet sand, damp and loose.

For brick work, —

1 cask lime, or 8 cubic feet stiff paste.
2 casks cement, $7\frac{1}{2}$ " " " "
$38\frac{1}{2}$ cubic feet sand, damp and loose.

In laying very fine hammered stone masonry at the same works, a paste of hydraulic cement, with a small proportion of fine sand, was used.

The hydraulic mortar used at Fort Richmond, as given by General Gillmore, whether for stone masonry or concrete, was made as follows: —

1 cask (308 pounds net) hyd. cem. powder = 3.70 to 3.75 feet stiff paste.

3 casks, or 12 cubic feet loose sand.

The result was 11.75 cubic feet of rather thin mortar.*

The mortar used at the Quincy Bridge consisted of one half Louisville, "Falls City," hydraulic cement, and half clean, sharp sand. The masonry at the Kansas City Bridge was laid in hydraulic mortar, of two parts sand and one part cement, except in the upper courses, which are rarely or never exposed to the water.

The crushing strength of Portland cement, as determined by Mr. Grant, is as shown below, the proportion of sand being varied from 1 to 5.†

Proportions.	Three Months	Six Months.	Nine Months.
Pure Cement,	3795	5388	5984
1 C + 1 S	2491	3478	4561
1 C + 2 S	2004	2752	3647
1 C + 3 S	1436	2156	2393
1 C + 4 S	1331	1797	2208
1 C + 5 S	959	1540	1678

The specimens were made into bricks $9 \times 4\frac{1}{4} \times 2\frac{3}{4}$, and pressed on the flat side. The blocks showed signs of failing with

* Practical Treatise on Limes, Hydraulic Cements, and Mortars. 1864.

† Proc. Inst. Civ. Eng, Vol. XXV., p. 97. Stoney, Vol. II., p. 218.

about $\frac{5}{8}$ of the load that finally crushed them. The above figures show the effect of the sand upon the strength of the mortar.

Concrete.

Concrete, or beton, is a mixture of mortar, commonly hydraulic, with some coarse material, as broken stone, brick, shells, or gravel. The product is an artificial conglomerate, which, when well made, has the strength of stone masonry, and is exceedingly convenient in many localities where large stones could not be laid. It is very largely employed in foundations, in the backing of walls, in the filling of piers, and it has recently been used for stone throughout entire structures. It may be laid down in horizontal beds over wide areas, or it may be made into blocks and used as stones.

The proportion of cement should be such as to form good mortar with the sand alone; and the mortar thus made should be somewhat in excess of the spaces to be filled, that the coarse material may be quite surrounded. The amount of mortar required will be reduced by the mixing of coarse and fine materials, the latter helping to fill the spaces between the former.

The concrete for the foundation of the sea wall at Lovell's Island, in Boston Harbor, was made as follows: —

Cement, 1 cask = 3.75 cubic feet stiff paste.

Sand, $10\frac{1}{4}$ cubic feet, damp and loose.

Result, 11 cubic feet of mortar, or $10\frac{1}{4}$ cubic feet, quite stiff, as used to make the concrete. With this was mixed $31\frac{1}{2}$ cubic feet of shingle from the beach, composed of pebbles of all sizes, from the bigness of peas to stones 6 inches in diameter. The concrete resulting made $33\frac{3}{4}$ cubic feet.

The foundation courses of a part of the scarp wall at Fort Warren rested upon a bed of concrete about a foot thick. The mortar was richer than that mentioned above, the proportions being, —

Cement, 900 pounds = $10\frac{1}{2}$ cubic feet, stiff paste.

Sand, 21 cubic feet, damp and loose.

The result was about 22 cubic feet of mortar.

In laying the pintle blocks for the Barbette guns on the cover face, where the ground was dryer, a portion of fat lime was used for the concrete, as follows: —

Lime, 1 cask = 8 cubic feet, stiff paste.

Cement, 975 pounds = 11.25 feet, stiff paste.

Sand, 42 cubic feet, damp and loose.

This produced 40 cubic feet of stiff mortar, with which was mixed $67\frac{1}{2}$ cubic feet of granite fragments and $33\frac{3}{4}$ cubic feet of beach gravel, making in all 100 cubic feet of the small stones, and with the mortar $115\frac{1}{2}$ cubic feet of concrete, weighing about 116 pounds per cubic foot.

The foundation concrete at Forts Richmond and Tompkins, New York Harbor, was composed of, —

Cement, 1 cask (308 pounds net) = 3.65 to 3.70 cubic feet, stiff paste.

Sand, 3 casks (or 12 cubic feet), loose.

The result was 11.75 cubic feet of rather thin mortar, to which was added 5 casks (20 cubic feet) of granite fragments, producing in all $21\frac{3}{4}$ cubic feet of concrete when rammed into the foundation. The concrete for the superstructure at the above works contained 11.75 feet of mortar and 16 of stone fragments. The foundation concrete contained about $\frac{1}{12}$ of its bulk of stones from $\frac{1}{4}$ to $\frac{3}{4}$ of a cubic foot, rammed into the heart of the wall as the concrete was laid.

The concrete at the Quincy Bridge consisted of 2 cubic yards of Quincy limestone, screened and broken to pass through a $2\frac{1}{2}$ inch ring, $3\frac{1}{2}$ barrels of "Falls City" (Louisville) cement, and $3\frac{1}{2}$ barrels of coarse river sand. When the broken stone was not screened the proportion of sand was diminished, the intention being to have one part cement, one sand, and four broken stone, by bulk.

At the Kansas City Bridge the beton consisted of 8 parts limestone, broken to pass through a 3 inch ring, 2 of sand, and 3 of cement.

It is an interesting fact that both masonry and beton were laid

in the above works in the severe winter weather by the use of hot sand and hot water. At the Quincy Bridge, during the coldest days each stone was held over a brazier of charcoal to draw out the frost. The mortar thus used was found the following spring to be as hard and perfect as any on the work.

The Breakwaters at Dover, England, and at Alderney, in the British Channel, have been largely built of blocks of concrete. At Marseilles, concrete blocks of 22 tons each, and of 353 cubic feet, were employed for the protection of the seaward slopes of the jetties. Blocks of the same size were used at the Mole of Algiers. The Cherbourg Breakwater, the parapet of which is 30 feet wide at the base, and 31 feet high, rests on a bed of concrete 7 feet thick, and is protected on the seaward side by concrete blocks containing 720 cubic feet each, and weighing 44 tons. The concrete foundation at one of the Graving Docks at Toulon is 400 feet long, 100 feet wide, and 15 feet thick, in a single mass.

The concrete at Fort Warren was prepared by first spreading out the gravel on a platform of rough boards in a layer from 8 to 12 inches thick, the smaller pebbles at the bottom and the larger on top, and afterwards spreading the mortar over it as uniformly as possible. The materials were then mixed by four men, two with shovels and two with hoes, the former facing each other, and always working from the outside of the heap to the centre, then stepping back and recommencing in the same way, and thus continuing the operation until the whole mass was turned. The men with the hoes worked each in conjunction with a shoveller, and were required to rub well into the mortar each shovelful as it was turned and spread, or rather scattered, on the platform by a jerking motion. The heap was turned over a second time in the same manner, but in the opposite direction, and the ingredients were thus thoroughly mixed, the surface of every pebble being well covered with mortar. Two turnings were usually enough to make the mixture complete, the resulting mass being $33\frac{3}{4}$ cubic feet of concrete, which was carried to the trench in barrows, and levelled, and well rammed in layers about a foot thick. The maul

was a cylinder of wood $8'' \times 8''$, with a handle, the bottom being faced with a piece of thick sheet iron. The concrete should be rammed until a film of water appears upon the surface, but not enough to make it quake.*

For making common mortar, the following method is suggested by General Gillmore, all of whose conclusions upon this subject are worthy of the fullest confidence of engineers.

"*First.* All the lime necessary for any required quantity of mortar should be slaked at least one day before it is incorporated with the sand. *Second.* The sand basin to receive the unslaked lime should be coated over on the inside with lime paste to prevent the escape of water. *Third.* All the water required to slake the lime to a stiff paste should be poured on at once. This will completely submerge the quicklime. The heap should then be covered over with tarpaulin or old canvas, and left till next day. *Fourth.* The ingredients should be thoroughly mixed, and the mortar heaped up for future use."

The mortar used at Forts Richmond and Tompkins, New York Harbor, for stone masonry or concrete, was made by hand, and composed of hydraulic cement and sand, without lime. The following description is from General Gillmore's work.

Each batch of mortar or concrete corresponded to one cask, or 308 pounds net, of hydraulic cement powder. *First.* The sand is spread in a rectangular layer 2 inches thick. *Second.* The dry cement is spread equally all over the sand. *Third.* The cement and sand are very thoroughly mixed by four men. *Fourth.* A basin is formed by drawing the material to the outer edge of the bed. *Fifth.* Water is poured into the basin thus made. *Sixth.* The material is thrown back upon the water, absorbing it, when the bed occupies the same space that it did in the beginning. *Seventh.* The bed is completely turned twice. If the mortar is for masons' use, it is heaped up, to be carried where required. If for concrete, the broken stone is spread equally over the bed, a bucket of water, more or less, according to

* Wright's Treatise on Mortars and Cements.

the absorbing power of the stone, and the temperature, is sprinkled over it, and finally the bed is turned and heaped up, the heaping being equivalent to a second turning.

It is stated by General Gillmore that recent experiments show that most American cements will sustain, without any great loss of strength, a dose of lime paste equal to that of the cement paste; while a dose equal to $\frac{1}{2}$ or $\frac{3}{4}$ the volume of cement paste may safely be added to any energetic Rosendale cement, without damaging the mortar to any great extent. This addition of lime does not unfit the mortars for submarine work, while we get a cheaper material, and by the use of the lime we prevent its setting too quick.

The "Beton-Coignet," used for very numerous purposes in France for some time past, is composed of 4 to 5 parts sand, 1 part lime, common or hydraulic, and from $\frac{1}{4}$ to $\frac{3}{4}$ of a part of cement, all in volume. This material is moulded into blocks, and possesses the characteristics of the softer stones, weighing 3700 pounds to the cubic yard, and having a crushing strength of from 2600 to 7500 pounds per square inch.

Mortar for Pointing and Flashing.

Pointing mortar, used to protect the joints of masonry laid in lime mortar, is made of a paste of finely ground cement and clean sharp sand, the volume of cement being a little in excess of the spaces in the sand, the bulk of sand being from $2\frac{1}{2}$ to $2\frac{3}{4}$ that of the cement. The mortar is mixed in small quantities, as needed, with but little water. The joint is cut out to a depth of an inch, carefully brushed clean, moistened with water, so that it may not take the water from the pointing, the mortar put in with a trowel, calked down with a hammer and calking iron, and finally well rubbed with a steel polishing tool In hot weather the joints should be kept moist, and, if possible, shaded from the sun. The mortar used at Fort Warren for pointing had 1 pound of cement powder to 3 pounds of dry sand; or by measure, 1 cement

paste and $2\frac{1}{2}$ dry sand. The mortar was made about a quart at a time, the ingredients being first mixed dry, and then placed in an iron mortar with a little water, and thoroughly incorporated by pounding with a heavy iron pestle. This compound is almost incoherent, and hardly plastic at all. The pointing will be done cheapest as soon as the masonry is laid, as the joints may then be very easily cleaned out before the mortar has become hard.

Flashing consists of a thin coat of cement mortar made with a very large part of cement. It is used to protect the face of walls exposed to the wet. Stone liable to disintegration may be protected by flashing.

Grout is a thin-tempered mortar, composed almost entirely of cement and water. It is run into voids in masonry which cannot be filled in the ordinary manner.

Structures of Masonry — Culverts.

The simplest form for a culvert for passing a small stream beneath an embankment is that shown in Fig. 139, consisting of two side walls and a covering of flat blocks of stone. The bottom

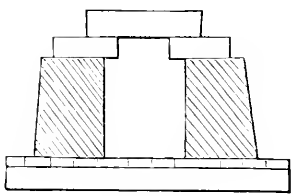


Fig. 139.

may be paved with stone, or the whole may be placed upon a platform of timber. Especial care should be taken in all works of this class to provide an ample way for the water to be passed, the amount of which may be ascertained from the area drained. The ends of culverts should be so arranged that the water can in no case work beneath or behind the masonry. The dimensions for such structures may be as follows : —

Area.	Opening.	Side Wall.	Depth of Covers.	Length of Covers.
4	2 × 2	2 × 2	12	5
9	3 × 3	3 × 2½	16	6
16	4 × 4	4 × 3	20	7
25	5 × 5	5 × 3½	22	8
36	6 × 6	6 × 4	24	9

Under high embankments the depth of cover may be increased at the middle of the length of the culvert, where the load is great-

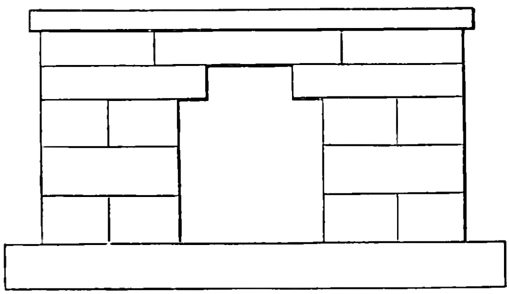


Fig. 140.

est. When appearance is to be regarded, the ends of the culverts may be finished with a head wall, as in Fig. 140. For larger struc-

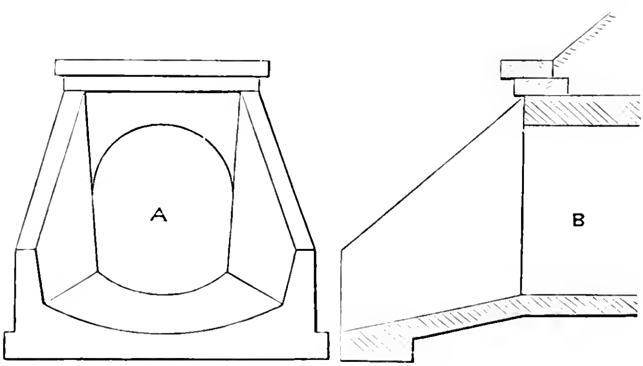


Fig. 141.

tures, the arch, Fig. 141, may be used ; A being an end elevation, and B a longitudinal section, through coping, arch, and floor. Fig. 142 shows a transverse section of the same through arch.

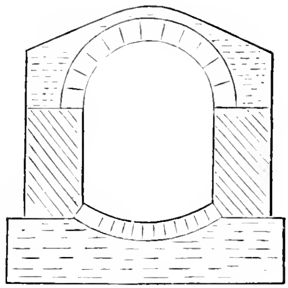


Fig. 142.

side walls, backing, and paving. For larger works of the same kind, or for passing a road beneath the railway, or even for crossing a small stream, either of the plans shown in Fig. 143 may be used.

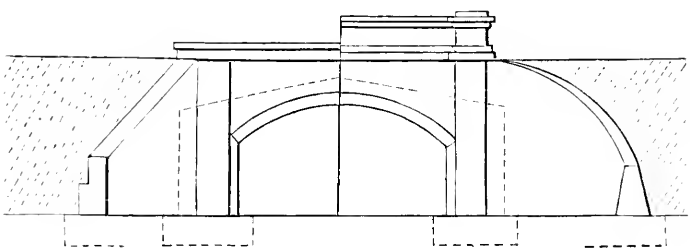


Fig. 143.

Retaining Walls.

A revetment, or retaining wall, made to sustain a mass of earth or water, requires a certain thickness to resist an overthrow. The mass of earth tends to assume its natural slope, and thus thrusts horizontally against the masonry. The amount of this thrust depends upon the height of the mass to be supported, and upon

the quality of the earth. The power of the wall to resist overthrow depends upon its thickness and upon the quality of the workmanship. A revetment may fail by sliding on its base, by revolving around its outer edge, or by a breaking or bulging of the body of the masonry. The second method is that commonly assumed to occur in the formulæ. The earth is supposed to act as at F, Fig. 144, with a leverage F B, and the wall to resist with its weight, and a leverage A C (C being a point in the base directly beneath the centre of gravity). If the front of the wall

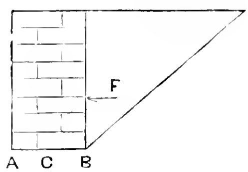


Fig. 144.

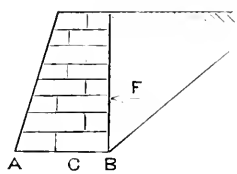


Fig. 145.

is battered, the leverage of resistance is of course increased, as seen in Fig. 145. If the earthwork rises higher than the top of the wall, of course the point of application of the overthrowing force is raised. The weight of the masonry is an element which admits of tolerably precise determination, as well as its leverage of resistance; but the amount of the overthrowing force is quite indeterminate. Experiments made upon certain masses of sand and gravel to determine their natural slope, and thence their thrusting force, are of little use when applied to the various qualities of material, often saturated with water, which are met with in practice. If we assume, as is generally done, the pressure of a mass of water to be so distributed as to be equal to a single force acting at a point one third of the depth from the bottom, and the amount to be equal to the area of surface multiplied by the depth of the centre of gravity below the level of the water, and by the weight of a unit of water, we get directly the overthrowing force which may be taken as a maximum, and therefore as giving safe results. The precise factor of safety to be adopted

is another question, which will vary in almost every special case. A wall to resist the action of a quiescent mass needs less strength than if it is intended to retain an embankment subjected to the weight and jar of a railroad train. The elaborate investigations of mathematicians upon this subject are more interesting to themselves than useful to engineers. The empirical rule of making the mean thickness of the wall $\frac{1}{3}$ the height for ordinary cases, and $\frac{1}{2}$ the height for surcharged revetments, and battering the front so that the top, middle, and bottom shall be as 3, 5, and 7, with masonry of a fair character, but not ham-

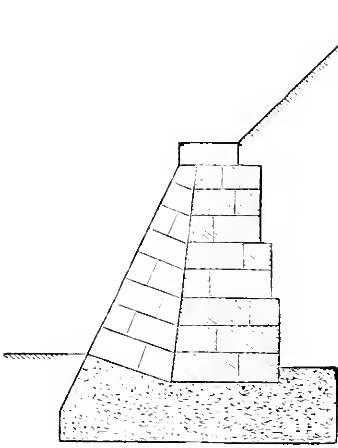


Fig. 146.

mered, has been found to give safe results.* In special cases of unusual difficulty it may be advisable to aid the retaining wall by buttresses, as in Fig. 146, or by counterforts, or by relieving arches behind the wall. Very much depends, also, upon the man-

* The rules given in some of the engineers' pocket manuals, in which the thickness of retaining walls is made from one fifth to only one seventh of the height, are entirely unreliable, and can lead only to failure. Mr. Trautwine's new pocket book contains, however, valuable and reliable memoranda upon this subject, so presented as to meet precisely the demands of the practical engineer.

ner in which the earthwork is arranged behind the masonry, and upon the care taken to secure thorough drainage. Every precaution should be taken to keep the water from working in between the masonry and the earth, and in cases where this cannot be done, provision may be made to collect the water, and lead it through suitable openings in the masonry to the front. By arranging the earthwork behind the masonry in well rammed layers, inclined downwards from the wall, the thrust may be much reduced. In the case of very high walls they may be made hollow, or a plain wall may be buttressed by arches in front, or relieved by arches behind. In the case of very low walls, where the weight of the masonry is but little, the thickness should be as much, or even more, than the height of the wall. In all cases the mortar should be allowed to get hard before the thrust is allowed to act upon the masonry, and especial care should be taken to secure a firm foundation.

Bridge Abutments.

The form of a bridge abutment will depend upon the locality, and upon the use to which the bridge is to be put, whether for a railroad or for common travel ; whether near a large city or in a location where appearances need not be regarded. Where a river acts dangerously upon a shore, wing walls will be necessary. These wings may be curved or straight, and may be simply the abutment produced, or may be swung around into the bank at any angle, until the winged abutment, shown in plan, at A B, in front elevation, at D E, and section C, Fig. 147, becomes the ⊔ abutment, Fig. 148, or, by moving the wings on to the centre line, takes the form of the ⊤ abutment, Fig. 149. The curved wing, being arched, requires a little less thickness, but at the same time is longer. The slope of the wings may be finished with an inclined coping, or offset at each course. Wing walls, subjected to special strains, or to particular currents of water, require positions and forms accordingly. In skew bridges the

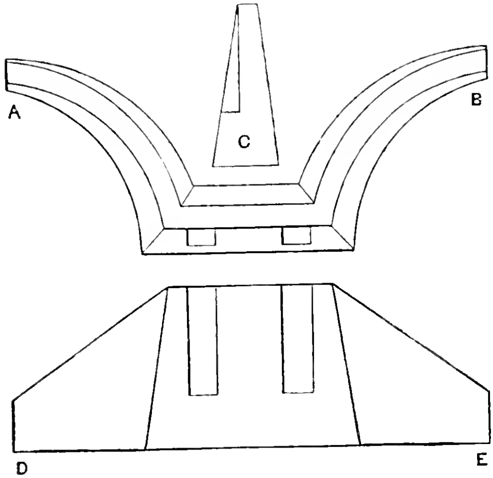


Fig. 147.

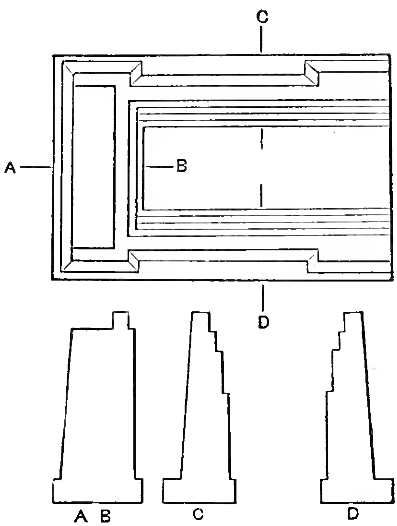


Fig. 148.

wing at the acute angle is longer, and inclines less from the face of the abutment than that at the obtuse angle. The more the wing

departs from the face line and swings round into the slope, the greater the thrust becomes upon it, since the centre of pressure is raised. The thrust becomes a maximum at an angle of about 45°, at which position the centre of pressure is about $\frac{2}{3}$ the height of the abutment. When the wing is thrown farther into the bank the slope begins to fill up in front of the wall, and to balance the pressure behind. The body of an abutment, as well as any other retaining wall, will be strengthened by a proper batter, thus en-

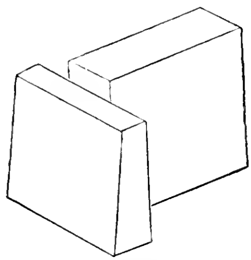


Fig. 149.

larging the base. Railway abutments, unless for a double track, require but little breadth on top, except where the truss rests. The common T abutment, Fig. 149, in which the stem takes the

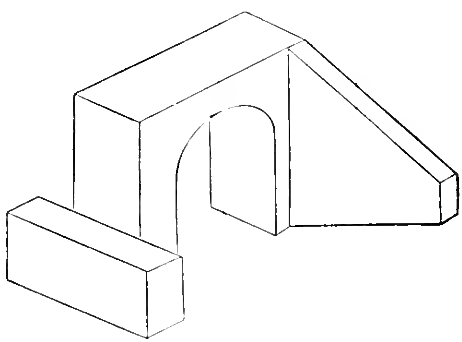


Fig. 150.

place of the wings, seems to fulfil every demand, where the bank does not require to be protected from the action of water. The difference of level of the top of the abutment and of the bridge seat

depends upon the difference between the height of the bearing of the lower chord of the bridge and the grade line of the road.

When an abutment becomes of considerable height, a saving may be made by lightening the T by an arch, as in Fig. 150. The dimensions of the bridge seat and of the T will, of course, be determined by the style of the truss, the gauge of the road, the height of the grade line, etc. The length of the T wall, measuring back from the front of the masonry, will depend upon the height of the abutment and upon the angle at which the embankment slopes. Those parts of the masonry opposed to the pressure of masses of earth are to be treated as retaining walls. Special strains, like those coming from the thrust of arches, may often be provided for by buttresses or counterforts placed as nearly as may be in the direction of the strain. The reader is especially referred for information upon the points mentioned above to Mr Trautwine's excellent Pocket Book for Civil Engineers.

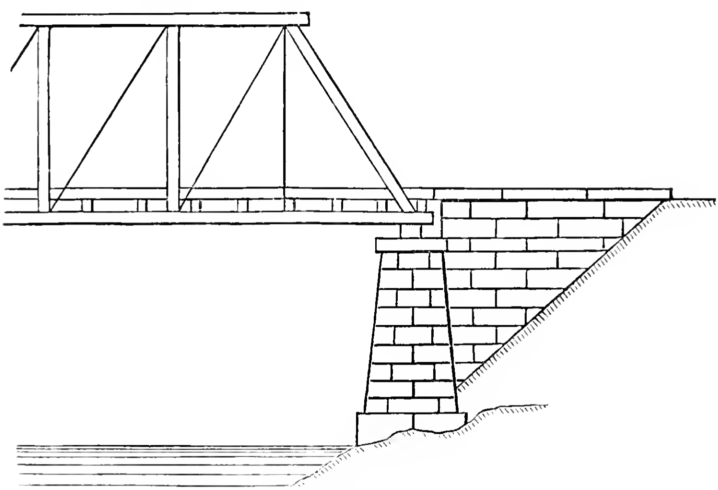


Fig. 151.

Fig. 151 shows the general arrangement of the T abutment for an overgrade bridge. For an undergrade truss the bridge seat would be lowered by an amount equal to the depth of the truss.

THICKNESS OF PIERS.

The thickness of a pier for simply supporting the weight of superstructure need be but very little at the top, care being taken to secure a sufficient bearing at the foundation. Piers should be thick enough, however, to resist shocks and lateral strains, not only from a passing load, but also from floating ice and ice jams, and in rivers, where a sandy bottom is liable to deep scouring, so that the bottom may wash out much deeper on one side of a pier than on the other, regard should be paid to the lateral pressure thus thrown on the pier. For mere bearing purposes the following widths are ample for first class masonry: —

Span	50	feet,	width	4	feet.
"	100	"	"	5	"
"	150	"	"	6	"
"	200	"	"	7	"

When the pier is to resist the thrust of arches, either of stone or wood, an additional thickness will be required.

The amount to which the water way of a stream may safely be contracted will, in some cases, determine the number and thickness of the piers. In a series of stone arches, unless the piers are made thick enough to resist the lateral thrust, it will be necessary to carry them all up at once, thus involving a considerable expense for centering. The up-stream end of a pier, subject to shocks from ice, or in swift water, requires to be sharpened so as to offer the least resistance possible to the moving masses. Six several examples of large piers in rivers, very different in their features, are given in Plate XXVII. The piers of the Kansas City Bridge are widened at the top to secure a broad seat for the bridge. The batter of the sides is $\frac{3}{4}$ of an inch to the foot, or 1 in 16, and the same on the starling. The ice-breakers have a batter of 6 inches in a foot, giving to the cutting edge of the nose a retreat of $8\frac{1}{2}$ inches in each vertical foot. The angle included by the faces of the starling on the plan, is 90°. The nose

of the ice-breaker is protected by a heavy plate of cast iron, and the shoulders were carefully dressed to a curve after the stone was laid. The specifications required the piers to consist of the best description of rock range work, the face stones to be cut, squared, and bedded with $\frac{1}{4}$ inch joints, and with the vertical joints cut back at least 9 inches from the face. The ice-breaker faces were to be cut smooth, and drafts cut on all angles. The shoulders and corners were to be trimmed so as to have no projection exceeding $1\frac{1}{4}$ inches, and no projection exceeding 4 inches was allowed on any part of the pier. The whole size of the top of each pier was finished smooth with a bush hammer. The face stones were fastened by cramps of inch round iron, as high as the tops of the ice breakers, and this system of dowelling was continued at the shoulders up to the overhanging courses, where it was again extended to the whole face. The backing was formed of heavy uncut stone laid in full mortar beds, the crevices being filled with small stones laid also in mortar. The whole amount of masonry was laid in hydraulic mortar, made of 2 parts sand to 1 cement, except the upper courses, which are laid in mortar mixed with a paste of fat lime.* The cost of the masonry was about $18 per yard.

The channel piers of the Quincy Bridge are made of rock-faced ashlar, backed with concrete, having a batter of $\frac{1}{2}$ inch per foot in the body of the pier, and of 3 inches per foot in the lower part. The upper end is finished to a right angle on the plan, the nose having a slope of 8 inches per foot, protected with a bar of iron. All of the stone was required to be perfectly sound and free from imperfections, one third of the blocks averaging over 16 inches in height, one third from 12 to 16 inches, and one third from 10 to 12 inches. The face stones were required to have not less than 10 square feet area of bed, and in each course there were to be four headers $2\frac{1}{2}$ feet long. The ice-breakers and cut-waters were to be formed of three stones only in each course, the courses breaking joint with each other not less than a foot. The coping

* The specification for masonry above is taken from "The Kansas City Bridge," already referred to.

was 15 inches thick, and long enough to cover the whole width of the pier. In the first pier the courses were alternately headers and stretchers. All stones were cut to lay on their natural beds, which were dressed true to a $\frac{3}{8}$ inch joint throughout. Vertical joints were dressed square for nine inches from the face. The whole upper face of the ice-breakers and the copings were bush-hammered, and a draft 4 inches wide was chiselled on all of the angles. The three stones at the nose and foot of the piers were cramped with iron bars $2 \times 1\frac{1}{2}$ inches, covered with coal tar, on each course; and every fourth course was cramped throughout. Upon the first pier every course was cramped. The cost of the above pier masonry was \$22.06 per cubic yard.

The piers of the Louisville Bridge, 26 in number, are of limestone, laid in hydraulic cement, and, with the exception of the two next the abutments, are founded upon the solid rock bed of the river. They vary in height from 40 to 100 feet. Up to high water they are finished at both ends with Gothic cut-waters, with constant radii of 8′ 6″ in the piers between the channels and shores, 9 feet in the piers between the channels, and of 12′ 6″ in the channel piers; the effect being to change constantly the angle of intersection at the point, this diminishing with the thickness of the pier. The piers between the channel spans on which the 245 feet trusses rest are 7×21 feet under the coping; those between the channel spans and the ends of the bridge 6×21. The channel piers are 10′ $4\frac{5}{8}$″ × 33′ $4\frac{5}{8}$″, besides the semicircular ends. The cut-water caps are all at the same level. The piers above the caps are carried up with a rectangular section, except the four on which the channel spans rest, these being finished with semicircular ends. The batter is in all cases 0.43 inches per foot. The piers are finished with bush-hammered cut-water caps, string courses and copings.

The channel piers of the new bridge across the Hudson, at Albany, are represented in Plate XXVII. The size under the coping is 7 × 39′ 10″. The form of the up stream end is shown in the drawing, the shoulders of the nose being rounded off.

Plate XXVII. shows also an elevation, plan, and arrangement

of stones on top of the guard piers to the pivot span of the bridge across the Susquehanna, at Havre de Grace, upon the Philadelphia, Wilmington, and Baltimore Railroad. These piers are subjected to very severe pressure from ice, and were the result of very careful study. The guard piers, 31 feet both in length and width, are placed 85 feet distant from the centre of the pivot pier, the latter being about 25 feet in diameter.

The following table gives the dimensions of the several piers for the fine bridge across the Missouri River, at St. Joseph.

No. of Pier.	Size at Top.	Size at Bottom.	Height from Bed to Top of Pier.
1	8 × 25 ft.	22′ 6″ × 38′ 6″	71 ft.
3	10 × 25 ft.	25′ 6″ × 56′ 8″	75′ 4″
4	9′ 1″ × 25 ft.	24′ 4″ × 53′ 3″	75′ 3″
5	9′ 1″ × 25 ft.	24′ × 56′	73 ft.
6	8 × 25 ft.	24′ 4″ × 38′ 4″	71 ft.

Forms for Stone Arches.

The form of an arch may be the semicircle, the segment, or a compound curve formed of a number of circular curves of different radii. Full centre arches, or entire semicircles, offer the advantages of simplicity of form, great strength, and small lateral thrust; but if the span is large they require a correspondingly great rise, which is often objectionable. The flat or segmental arch enables us to reduce the rise, but it throws a great lateral strain upon the abutments. The compound curve gives, when properly proportioned, a strong arch, with a moderate lateral action, is easily adjustable to different ratios between the span and the rise, and is unsurpassed in its general appearance. In striking the compound curve, the following conditions are to be observed: The tangents at the springing must be vertical, the tangent at the crown horizontal, and the number of centres must be uneven. In the cases occurring upon railways, curves of 3 and 5 centres will be found to fulfil all requirements, though Perronet's

fine bridge of Neuilly, over the Seine, has 11 centres, the span being 128 feet, and the rise 32 feet.

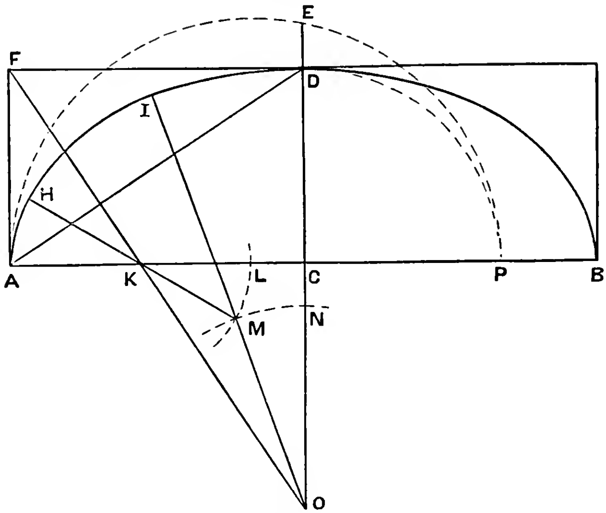


Fig. 152.

The five-centred curve is described as follows:—

Let A B represent the span, and C D the rise. Join D and A. Draw F O perpendicular to A D. Make C P equal to C D, and describe a semi-circle on A. P. Make C N equal to E D, and describe the arc M N. Make A L equal to C E, and from K as a centre describe the arc L M, cutting M N at M. K, M, and O will be the centres, and A K, H M, and I O the radii for half the curve. The other half is to be constructed in the same manner. This curve does not differ essentially from the ellipse.

Depth of Key Stone.

The depth of the arch stone, or thickness of voussoir, depends upon the form and size of the arch, the character of the masonry, and the quality of the stone. The following table gives the depths for small semicircular arches, the second column being for hammer-dressed beds, and the third column for beds roughly dressed with the chisel:—

Span in Feet.	Depth of Arch in Inches, 1st Class Masonry.	Depth of Arch in Inches, 2d Class Masonry.
6	12	15
8	13	16
10	14	17
12	15	19
14	16	20
16	17	21
18	18	23
20	19	24
25	20	25
30	21	26
35	22	28
40	23	29
45	24	30
50	25	31

Professor Rankine remarks very correctly, that the precise determination of the depth of the key stone of an arch would be an almost impracticable problem from its complexity, and that the best course in practice is to assume a depth for the key stone according to an empirical rule founded upon the dimensions of good existing examples of bridges. For such a rule he gives the following: — *

Depth in feet $= \sqrt{}$ (.12 radius at crown) for a single arch.

Depth in feet $= \sqrt{}$ (.17 radius at crown) for an arch of a series.

Mr. Trautwine gives the following rule.† For first class cut stone of hard material take 0.36 of the $\sqrt{}$ of Radius at the crown; for second class work .40 $\sqrt{}$; and for brick or rubble arches

* Civil Engineering. p. 425. Fifth Ed. 1867.

† Journal Franklin Inst., Vol. XL., 3d Ser., 1860, p. 233.

0.45 of the $\sqrt{}$. The results by the latter are slightly in excess of those by Professor Rankine's formulæ.

Mr. Trautwine illustrates his rule by the following valuable table:

Structure.	Engineer.	Span in Feet.	Rise in Feet.	Radius at Crown in Feet.	Actual Key in Feet.	Calculated Key in Feet.
Reading Railroad, ...	Steele, ...	31	5	26¾	1.66	1.86
James River Aqueduct, ...	Ellet, ...	50	7	47	2.66	2.47
Lugar Viaduct (Scotland), . . .	Miller, ...	50	25	25	2.00	1.80
Monocacy Aqueduct, ...	Fisk, ...	54	9	50	2.50	2.55
Bow Bridge, ...	Walker & Burges,	66	13¾	81	2.50	3.24
Staines Bridge, ...	Rennie, . . .	74	9½	78	3.00	3.18
Licking Aque., Ches. & O. Canal,	Fisk, ...	90	15	76	2.83	3.14
Pont Napoleon, Paris,*	Couche, ...	115½	14¾	120	4.00	4.93
Neuilly,†	Perronet, . . .	128	32	160	5.30	4.55
Maidenhead Viaduct,‡	Brunel, ...	128	24¼	169	5.25	5.85
Dora, Turin, ...	Mosca, ...	148	18	160	4.80	4.55
Gloucester Bridge, Severn, . .	Telford, ...	150	35	150	4.50	4.41
New London Bridge, Thames, .	Rennie, ...	152	29½	162	4.75	4.58
Grosvenor Bridge, Dee, ...	Harrison, . . .	200	42	140	4.00	4.26
Cabin John Aqueduct, ...	Meiggs, ...	220	57¼	134¼	4.16	4.17

The following additional examples are given by Professor Rankine, the calculated depths being by his formula given above:

St. Maxence, Segment, ...	Perronet, ...	76¾	6½	119	4.79	4.49
Waterloo Bridge, Ellipse, . . .	Rennie, ...	120.0	32.0	112½	5.00	4.37
Ballochmyle, Ayr, Semicircle, .	Miller, ...	181.0	90½	90½	4.50	3.92
Dean Br., Scotland, Segment, .	. . . .	90.0	30.0	48¾	3.00	2.88

* Small rubble in cement. Formula .45 $\sqrt{R}$.

† This bridge as designed had a radius at crown of 160 feet; but the arch settled so as to increase it to 250 feet.

‡ Brick laid in cement. Formula .45 $\sqrt{R}$.

Thickness of Abutments.

Numerous rules have been given for obtaining the thickness of the abutments for arches. The most elaborate of these are from their form applied with difficulty to the cases commonly occurring in practice, and many of the elements entering into the solution of the problem are quite indeterminate, depending as they do upon the character of the masonry and upon the workmanship. In place of rules, therefore, we present merely an empirical table, embracing the results of a considerable degree of practice.

Span.	Thickness of Rectangular Abutments for Semi-circular Arches, height in Feet being,			
	10	15	20	25
10	5	6	7	8
15	$5\frac{1}{2}$	$6\frac{1}{2}$	$7\frac{1}{2}$	$8\frac{1}{2}$
20	6	7	8	9
25	$6\frac{1}{2}$	$7\frac{1}{2}$	$8\frac{1}{2}$	$9\frac{1}{2}$
30	7	8	9	10
35	$7\frac{1}{2}$	$8\frac{1}{2}$	$9\frac{1}{2}$	$10\frac{1}{2}$
40	8	9	10	11
45	$8\frac{1}{2}$	$9\frac{1}{2}$	$10\frac{1}{2}$	$11\frac{1}{2}$
50	9	10	11	12

The above dimensions are for rectangular abutments; such, however, are seldom used, the common practice being to batter them on the back from $\frac{1}{6}$ to $\frac{1}{10}$. In such cases the above dimensions may be taken as the mean thickness of the abutment, which will increase the strength somewhat over that of the rectangular form.

Oblique Arches.

Skew, or oblique arches, are rarely employed in the United States. When they are used, they are made, not after the European method, with spiral or winding joints, but in separate ribs ; each rib being placed a little in advance of its neighbor, the result being an oblique arch. The invention of this method is due to B. H. Latrobe, the elder, who designed and built several such arches of brick, both in this country and in England. The first ribbed arch for railway traffic in America, was that built in 1848, upon the Philadelphia and Reading Railroad, at "Third Crossing," being of stone, with three spans of 42 feet, and 12 feet rise. Each arch consists of four square ribs set back $1\frac{1}{4}$ feet, and having thus a skew of 5 feet. At Latrobe, in Westmoreland County, Pennsylvania, the Loyalhanna is crossed by a stone bridge of three segmental arches, 45 feet span and 10 feet rise, the skew being made by building each arch of four ribs, which are right arches $5\frac{1}{2}$ feet wide, overlapping two feet. The "Falls Bridge," built of stone, and completed in 1856, over the Schuylkill River, on the Philadelphia and Reading Railway, has six arches, each having a span of 83 feet, and a rise of 24 feet. Each vault consists of eight square built ribs 3 × 3 feet in section, each rib being 18 inches back of its neighbor, making a skew of 12 feet in a width of 24 feet. The tops of the adjoining ribs in such arches may be connected above the voussoir, and a good bond should be secured in the spandrels.

CHAPTER XVI.

SUPERSTRUCTURE.

THE following statement, showing what the superstructure of a railroad should be, is taken from a valuable paper read before the Institution of Civil Engineers in 1852, by the distinguished English engineer, William Bridges Adams.

"The principal requirements of permanent way are, — That it be well drained, and especially in contiguity to the substructure. That the weight and damaging power of the engines and rolling stock should be considered as the datum for calculation. That the strength, hardness, and tenacity of the rails and the immobility of the substructure should be adapted for the hardest work to which the railway is to be subjected. That the substructure should have an amount of bearing surface proportioned to the load to be borne and the nature of the soil or ballast, and a sufficiently firm hold in the ground to prevent looseness or lateral movement from the side lurches of the engines or trains. That the rails should possess so much vertical and lateral stiffness, either in themselves or by their fastenings, as to prevent all deflection, and have sufficient hardness of surface not to laminate or to disintegrate beneath the rolling loads, and have sufficient breadth or tread-surface to diminish the effect of the crushing power of the wheels. They should be as smooth as possible on the running surface, to prevent concussion, and be laid at the proper angle, and the curves regularly bent so as to insure the accurate tread of the wheels, whilst the joints should be so made that the rails may practically become continuous bars, yet with freedom to expand and contract without being too loose. And

with all this there should be interposed between the rails and the solid ground some medium sufficiently elastic to absorb the effect of the blows of the wheels without being crushed or forced down into the ballast, and yet stiff enough to keep the upper surface of the rails in a uniform plane."

The object of the ballast is to transfer the applied load over a large surface, to hold the timber-work in place horizontally, to carry off the rain water from the superstructure, and to prevent freezing up in winter, to afford means of keeping the ties truly up to the grade line, and to give elasticity to the road bed. The material most commonly used for ballast is gravel, or gravel and sand, and where these cannot be obtained, broken stone, burnt clay, cinders, shells, and even small coal, have been employed.

The material should in every case be clean and hard, so as not to pack into a solid mass, thus preventing the passage of water away from the track. The common width on top of the ballast for the 4′ 8½″ gauge in England, with cross ties 9 feet long, is 14 feet for a single track, and 26 feet for a double track. The depth is generally 2 feet, one of which is below the tie, the latter being entirely covered by the ballast. In this country it is customary to sink the ties about half their depth into the ballast, in which case a depth of 16 inches in all is the least that will give a proper bearing beneath the tie. The quantity of ballast per mile required for different depths and widths is given in the following table: —

Depth in Inches.	Cubic yards of Ballast per Mile, the side Slopes being 1 to 1, and the width on top being, — For Single Track			For Double Track		
	10 feet.	11 feet.	12 feet.	21 feet.	22 feet.	23 feet.
16	2,955	3,216	3,477	5,823	6,084	6,344
18	3,374	3,667	3,960	6,600	6,894	7,187
20	3,802	4,128	4,454	7,387	7,713	8,039
22	4,242	4,600	4,959	8,186	8,544	8,903
24	4,693	5,084	5,475	8,996	9,387	9,775

THE CROSS TIES.

The timber work supporting the rails consists either of cross ties of wood, hewn flat on the top and bottom, from 7 to 9 feet long for the 4′ 8½″ gauge, 6 inches deep, and from 6 to 10 inches wide. Longitudinal sawed timbers, rectangular in section, placed beneath the rail, and giving it a continuous bearing, have also been employed, but have not been adopted to any great extent, possessing no advantage over cross ties, and being subject to some decided disadvantages. A continuous bearing is no better than a broken one for support, as the strength of the timber offers very little resistance to the weight of a locomotive. The strength is in the rail, the timber being employed as a means of keeping the rails in the proper position, and as an elastic medium between the rail and the ground. In the case of removal, with a continuous bearing, two rails at least must be taken up to admit of replacing a timber, while with cross ties any one may be taken out and replaced without interrupting the passage of trains. The distance at which the ties should be placed depends upon the weight of the engines traversing the road, and upon the strength of the rail. The timber should give a sufficient bearing between the rail and the wood, and also between the wood and the ground. The amount of the bearing depends upon the width and the distance of the ties. The latter should not be so little as to prevent the proper tamping of the earth around the tie. The amount of bearing surface per lineal foot of track for different distances and widths of ties is shown below:—

Distance Centre to Centre of Tie in Feet.	Square Feet of Bearing of Tie on the Ground per lineal Foot of Railway, the length being 8 Feet and the width, —				Ties per mile.
	6 inches.	7 inches.	8 inches.	9 inches.	
2	2.00	2.33	2.67	3.00	2,641
2¼	1.78	2.07	2.37	2.67	2,348
2½	1.60	1.87	2.13	2.40	2,113
2¾	1.45	1.70	1.94	2.18	1,921
3	1.33	1.56	1.78	2.00	1,761

A common practice is to make the distance 2 feet 6 inches from centre to centre, the tie being 8 feet long and 8 inches wide ; and with sufficient ballast and a 60 pounds rail, the result is good. It is very important that the size of the sleepers and the spaces between them should be uniform, in order that the bearing upon the ground may be equal at all points. Chestnut, oak, locust, hemlock, pine, and cedar are employed for ties. Spikes hold from two to three times as much in hard as in soft wood. In hard wood the fibres are driven back upon themselves and condensed, while in soft wood they are bent, broken, and destroyed. The light and inferior woods take up the various preservative solutions more readily than the harder kinds. The ordinary life of ties is from 5 to 10 years. Hemlock and spruce rarely last over 4 or 5 years. Cedar will last 8 or 10 years if it was not for the wearing of the rail at its bearing into so soft a wood. Oak and other good hard woods will last from 7 to 10 years. In Europe ties are nearly always preserved by some of the solutions referred to in Chapter VII., in consequence of which their life is prolonged to from 12 to 15 years.

An exceedingly simple but effective method of increasing the life of soft wood ties has been introduced by Mr. F. H. Whitman, which consists in the insertion of hard wood blocks into soft wood ties ; by which a hard wood bearing for the rail is combined with the lightness and cheapness, and in the case of cedar, the durability of the softer woods. Recesses 1½ inches deep and 10 inches long are accurately cut by gauge in the ties at the bearings of the rails, and the hard wood blocks, 10 inches long, 8 inches wide, and 1½ inches thick, also cut and bored for the spikes by gauge, are driven tight into the recesses, the grain of the hard wood running lengthwise of the tie, by which any loosening from shrinkage is avoided. The top surfaces of the blocks are thus brought invariably into the same plane, and the gauge of the road is truly defined by the holes for the spikes. Trials have shown that double the force is required to draw spikes passing through the hard wood blocks, to that needed to draw them out

of a soft wood tie. Longitudinal sections, made through a variety of ties, thus treated, have shown the fibre of the soft wood ties to be almost entirely destroyed, except about an inch in depth at the top surface, while with the hard wood blocks not only does the spike have a firm hold in the hard wood, but the soft wood beneath is much less damaged than when the block is absent. It may be safely stated that ties prepared as above are in every respect as good as those entirely of hard wood. The superior lateral stability given to the track by the hold which the spikes have in the hard wood, is shown by a sharp curve at Portland, Maine, the radius being 396 feet, and the length of curve 1000 feet. Over this track the heaviest palace cars and a large traffic are drawn by powerful engines, and while the sides of the rails are very rapidly cut up, the gauge remains perfect, with no more than ordinary spiking, viz., four spikes to a tie. By means of the superior bearing of the rail upon the tie afforded by the hard wood block, if the bearing between the ties and the ballast is sufficient, the ties may be placed farther apart than when the bearing is upon the soft wood of the tie. Upon a portion of the Maine Central Railroad the number of ties has thus been reduced from 2640 to 2420, saving 220 ties and 880 spikes per mile, and the track is unusually firm and uniform at this point.

Rails.

A good rail must be able to act as a girder or support between the ties, and as a lateral guide for the wheels. It must possess a top surface of sufficient size and hardness to withstand the action of the rolling weights, and a bottom surface which shall afford the requisite amount of bearing upon the tie. One square inch of rail section weighs 10 pounds per running yard, very nearly, which, at $60 a ton for iron rails, would amount to $1000 per mile of track ; whence the need of rolling the rail to that form which shall give the required strength with the least weight, may plainly be seen. The stiffness of rectangular beams is as

the cube of the depth. If the rails deflect, the result is the same as if the wheels were rolling up a grade. Mr. Barlow found the deflection of a 50 pounds rail $3\frac{5}{8}$ inches deep, the bearing being 3 feet, under a load of 5 tons at rest, to be equal to a grade of 25 feet per mile. At high speeds it would be more. If from settling of the track any one sleeper does not bear upon the ground, this deflection will, of course, be greater, corresponding to a steeper grade. The importance of depth in a rail is now fully seen, the old heights of $3\frac{1}{2}$ and $3\frac{3}{4}$ inches having given place to 4, $4\frac{1}{2}$, and 5 inches. The old idea that the head needed to be supported from the stem by a long slope is now seen to be erroneous. The head may be as little as an inch deep, connected with the stem by a short curve, thus allowing room for a good fish plate. The edge of a rail well made, and of good material, never splits off. The old 45 pounds Reading rail had a head $2\frac{1}{4}$ inches wide and $\frac{5}{8}$ of an inch deep only, and it stood an immense traffic for 20 years. On the Boston and Lowell Railway trains were run four years on the lower flange of the common American rail, the very wide and thin edge having been permanently bent down, but not broken.

From 60 to 65 pounds per yard appears to be sufficiently heavy for any service, provided the rail is well proportioned. The iron is much better worked in a light than in a heavy rail, and while the extra working adds to the cost per ton, the durability is increased in a much greater ratio.

The form of rail in general use in America, and indeed everywhere, except in Great Britain, is that shown in Plate XXVIII., Fig. 1, which is the 65 pounds steel rail for the Pennsylvania Railroad. The rail is $4\frac{1}{2}$ inches high, and 4 inches wide at the foot. The width of the head varies from $2\frac{1}{4}$ to $2\frac{1}{2}$ inches; the thickness of the stem from $\frac{1}{2}$ to $\frac{3}{4}$, though recent experiments indicate even a less thickness than the above as sufficient.*

* In 1869, Baron Von Weber, Director of the State Railways of Saxony, published at Weimar a valuable work upon the "Stability of the Permanent Way of Railways," from which the following memoranda are taken: —

Sandberg's Standard Rail Sections.

It has been suggested, and the suggestion is a good one, that a series of standard sections for rails of different weights should be adopted, both for the convenience of the railroad companies and the manufacturers. Such a system has been prepared by Mr. C. P. Sandberg, the inspector in Great Britain for the Swedish and Norwegian government railways.

The following are the sections proposed by Mr. Sandberg, the general form being that shown in Fig. 2, Plate XXVIII., the dimensions being in inches: —

No.	Weight in Pounds per Yard.	Depth.	Width of Head.	Thickness of the Stem.	Diam of the Bolt.	Thickness of Fish Plate.	Depth of Fish Plate.
1	40	$3\frac{1}{4}$	$1\frac{7}{8}$	$\frac{15}{32}$	$\frac{3}{4}$	$\frac{5}{8}$	$1\frac{13}{16}$
2	45	$3\frac{1}{2}$	$1\frac{31}{32}$	$\frac{1}{2}$	$\frac{3}{4}$	$\frac{3}{4}$	$1\frac{15}{16}$
3	50	$3\frac{3}{4}$	$1\frac{31}{32}$	$\frac{1}{2}$	$\frac{3}{4}$	$\frac{13}{16}$	$2\frac{1}{8}$
4	55	4	$2\frac{1}{8}$	$\frac{1}{2}$	$\frac{7}{8}$	$\frac{13}{16}$	$2\frac{1}{4}$
5	60	$4\frac{1}{4}$	$2\frac{3}{16}$	$\frac{1}{2}$	$\frac{7}{8}$	$\frac{13}{16}$	$2\frac{7}{16}$
6	65	$4\frac{1}{2}$	$2\frac{5}{16}$	$\frac{1}{2}$	$\frac{7}{8}$	$\frac{29}{32}$	$2\frac{5}{8}$
7	70	$4\frac{3}{4}$	$2\frac{3}{8}$	$\frac{9}{16}$	1	$\frac{15}{16}$	$2\frac{7}{8}$
8	75	5	$2\frac{7}{16}$	$\frac{9}{16}$	1	$\frac{15}{16}$	$3\frac{1}{8}$
9	80	$5\frac{1}{4}$	$2\frac{1}{2}$	$\frac{9}{16}$	1	$\frac{31}{32}$	$3\frac{1}{4}$

"The experiments made in 1851, by Weishaupt, to determine the best section of rail for the Eastern Railway of Prussia, and those of Malberg, made in 1857, for the Lower Silesian Railway, show, — That the flat-footed, or American rail, is, in all respects, the best form. That fibrous wrought iron rails offer about the same resistance to compression as to extension, and thus that the neutral axis passes very nearly through the centre of gravity of the section. That the limit of elasticity is the highest in rails having a base of fibrous and a head of crystalline iron. In 1858 Baron Von Weber proposed to the Saxon government to reduce the thickness of the stem from $\frac{5}{8}$ to $\frac{1}{2}$ inch, and by adding the metal thus saved to the height, to increase the strength by

The above depth of the fish plate is measured at the middle of its thickness. It will be seen that for every $\frac{1}{4}$ inch of increase in the width and height, the weight increases by 5 pounds. The flange is made $\frac{5}{16}$ of an inch thick at the edge, and the angle of its upper face with the under side is 11°, the under side of the head having a similar inclination, so that the fish plate may be reversible. The lower corners of the head are curved to a radius of a fourth of an inch; the top is curved to a radius of 6 inches; the sides and top of the head are connected by curves of a half-inch radius; the curve between the stem and the head and foot has a radius of $\frac{3}{8}$ of an inch; and the outer edge of the flange a radius of $\frac{1}{8}$ of an inch. The holes in the rail are $\frac{1}{4}$ of an inch larger, and those in the fish plate $\frac{1}{8}$ of an inch larger than the bolt. The centre of the first hole is $2\frac{3}{16}$ inches from the end of the rail; the distance from centre to centre of holes $4\frac{1}{2}$ inches, from the end of the fish plate to the centre of the first hole in the plate $2\frac{1}{4}$ inches. The notches for the spikes are $\frac{3}{4}$ of an inch long and $\frac{3}{8}$ of an inch deep 12 inches from the end of the rail on one side, and 10 inches on the other. The spikes are half an inch square, and $4\frac{1}{2}$ inches long, for 40, 45, and 50 pounds rails; $\frac{9}{16}'' \times 6$ inches for 55, 60, and 65 pounds rails, and $\frac{5}{8}'' \times 6$ inches for 70, 75, and 80 pounds rails.

The dimensions given above are very good, except that the foot is made rather wide, being in each case equal to the height of the rail. This gives a large and easy bearing upon the tie, which is, of course, well; but for light rails the extra material will do more service if put into the stem to increase the height. The general practice seems to show that a width somewhat less than the height is ample.*

20 per cent. He reduced the stem from 0.59 inches to 0.12 inches, and still found it amply stiff, and concluded that webs $\frac{3}{8}$ or $\frac{1}{2}$ inch thick are amply strong for rails of ordinary height, and that webs should be made as thin as the process of rolling, and the provision for sufficient bearing for the fish bolts, will permit.

* From the experiments made in 1851, for the Prussian government by Weishaupt, it is concluded that as the outer edges of the flat foot do not follow

If the top surface of the rail is made perfectly flat, and the wheel tire does not fit it exactly, from want of the proper position of the rail by settling, or from any cause, the wheel will bear entirely upon one edge, thus wearing the rail unequally. To remedy this a slight convexity is given to the top. Mr. Clark (in Railway Machinery) recommends the top to be curved to a radius of 10 or 12 inches. Mr. Sandberg's sections are curved to a radius of 6 inches. The Pennsylvania Rail, Plate XXVIII., Fig. 1, has a top radius of between 9 and 10 inches.

Inclination of Rails.

As the tread of the wheel is conical, the top of the rail must be inclined to fit this cone, otherwise the wear will come upon the inner edge of the rail only. This may be done in two ways: by placing the rail base level, and inclining the vertical axis of the cross section of the rail, and making the tread square with that axis; or by making the rail section true, and inclining the base, either by cutting the tie or by using a block like that of Mr. Whitman's, let in at the proper angle. The amount of cant or bevel to be given to the rail depends upon the cone of the wheel,

the vertical deflections produced by heavy pressures, the width and thickness must be kept within certain limits. The rail sections adopted by the Cologne Minden Railway Company successively, after careful examinations, show a continually decreasing width of base, as below: —

Dimensions and Proportions.	Section of 1844	Section of 1849.	Section of 1852.
Height of Rail, ...	3.66	4.25	4.75
Width of Base, ...	3.86	3.67	3.50
Width of Head, ...	2.32	2.33	2.25
Thickness of Stem, ...	0.625	0.75	0.542
Weight per yard, ...	57. lbs.	72. lbs.	65.25 lbs.
Proportion between weight and strength,	1.0	1.6	2.0

By the last change, while the depth was increased by 15 per cent., and the strength by 25 per cent., the weight was reduced by about 10 per cent. by reducing the width of the head and the base.

and this latter depends upon the gauge of the track and the radius of curvature. Upon the Prussian railways, gauge 4′ 8½″, the rails are inclined $\frac{1}{20}$, or 5 per cent. of the height, irrespective of the rail base ; i. e., for a rail 5 inches high the sleeper would be notched ¼ inch.

Duration or Life of Rails.

The duration or life of rails depends upon the quality of the iron of which they are made, and upon the amount and speed of the traffic passing over them. The rails first made in England were rolled from best No. 1 cold blast mine iron, run out in a finery fire, puddled, shingled, rolled into bars, which were cut, piled, heated, and hammered into blooms, and these were reheated and rolled into rails. The result was a material such as is rarely seen upon our railways at the present time. The old 45 pounds Reading rail bore an immense traffic for 20 years. The iron at first used upon English roads in many cases withstood from 15 to 20 years of wear under an enormous traffic. It was estimated in 1849, upon the London and North Western Railway, that 50 trains a day for 20 years, or 313,000 trains, would wear out a 70 pounds rail. With the material now put into railway tracks, however, no such service is obtained. Under favorable conditions, *good iron* will last 8 years under an average traffic ; but with the material commonly employed, from 3 to 6 years is all that can be counted upon for roads doing a good amount of work ; and the life of the iron upon the great routes is in many cases less than this. Mr. Price Williams states* that the average life of the best quality of iron rails has been found to be, in round numbers, 15,000,000 tons, or 100,000 trains of 150 tons each. That where from 12,000,000 to 15,000,000 tons of moderately fast traffic will wear out an iron rail, 38,000,000 tons of slow traffic are required to do the same. That the wear is greatest upon grades falling

* Maintenance and Renewal of Permanent Way. Paper read before the London Inst. Civ. Eng., March, 1866.

in the direction of the traffic. A mixed traffic of 62,399 trains, or of 12,000,000 tons, very nearly, wore out the best rails on a 40 feet descending grade in $7\frac{1}{4}$ years, while the same description of rails upon a level, under a very heavy freight traffic, required 203,122 trains, or 38,000,000 tons, very nearly, to wear them out. For equal amounts of traffic, the wear of rails is assumed to be as the square of the speed.

With regard to the life of steel rails, everything here again depends upon the quality of the metal. A pair of 21 feet Bessemer steel rails, upon the London and North Western Railway, outlasted 16 faces of iron rails, being evenly worn to a depth of a little over $\frac{1}{4}$ of an inch; having in about 3 years sustained the traffic of upwards of 9,000,000 engines and cars, or about 95,000,000 tons. So, too, upon the Philadelphia, Wilmington, and Baltimore Railroad, a good steel rail has outlasted 16 iron rails, the use of both being the same.

Iron and Steel Rails.

Whether it will be the most economical to employ rails of iron or steel, or of iron with a steel head, depends upon the amount of traffic which is to pass over the track, and the facility for obtaining the different materials. The interest upon the first cost of rails is an important element in this question. If a good iron rail will last 10 years, and interest is taken at 6 per cent., iron would be more economical than steel; but when the traffic is great enough to wear out an iron rail in 4 or 5 years, steel will be preferable.* It will thus often be most economical to employ iron upon some parts of a road and steel upon others. The following considerations upon this matter have been presented by a prominent railway manager, who has given especial attention to the subject.†

* The President of the Philadelphia, Wilmington, and Baltimore Railroad states, that on that road where iron does not last 7 years on an average, true economy, as regards the rail expenditure alone, demands steel rails.

† Ashbel Welch, Esq., President of the Camden and Amboy Railroad.

"The economic value of a short-lived superstructure is most affected by endurance; that of a long-lived one, by interest, on which depends the present values of future quantities. If one mile of rails will last one month, and another ten months, on the same road, and the cost of renewal in each case is equivalent to a total loss of the rails, then, for that road, the latter is worth nearly ten times as much as the former. But if one will last ten years, till money doubles, and the other ten times as long, then the latter, for that road, is not worth quite twice as much as the former.

"The present value of the second decade of the lifetime of a rail, is only half that of the first; of the third decade, only a quarter, and of the tenth, less than a five-hundredth part that of the first.

"On a road where iron rails will last six or eight years, it is, therefore, of comparatively little consequence whether steel will last half a dozen years, or a dozen times as long.

"As the value of the rails for a particular road depends not only on their endurance, but also on the amount of traffic they are to carry in some specified time, that is, on their opportunity for usefulness, it is important to estimate correctly, not only what they are able to carry, but what there will be for them to carry. An error in estimate of traffic on any road, is also an error in estimate of value of rails for that road.

"It greatly facilitates the comparison of the values of rails, or other things of different duration, with constant traffic or constant tendency to deterioration, to compare both with those that under the same circumstances will last forever. The relations of the destructibles to the indestructible are simpler than their relations to each other.

"The practical question has generally been, and often continues to be, not which is most economical for perpetuation, but whether iron should be used first, while traffic is light and money scarce and steel dear, and then, when worn out, be replaced by steel."

In order, therefore, to determine the relative value of iron and

steel for any particular case, we have not only to know the prices and the exact quality of the two metals, but also the financial conditions under which they are to be paid for, and the amount of traffic which may be expected to pass over them. Besides the matter of endurance, it is also to be considered that the steel rail deflects much less under a given load, and thus reduces the resistance to traction. Thus the mean of 82 experiments upon 68 pounds Ebbw Vale Company's iron rails, and the mean of 45 experiments upon 68 pounds steel rails, by John Brown & Company, Atlas Works, Sheffield, compared as follows: —

Material.	Load 20 Tons.		Load 30 Tons.	
	Deflection. Inches	Permanent Set. Inches.	Deflection. Inches.	Permanent Set. Inches.
Iron, ...	0.72	0.58	2.13	1.96
Steel, ...	0.32	0.15	1.72	1.20

Finally, the employment of steel rails, by giving a smoother and better track, reduces the cost of repairs of the rolling stock, as the track and machinery act and react upon each other, and thus adds to the safety of travelling.

Especial care is needed in the making of steel rails, that they should be straightened before they become cool, many failures having occurred at the point where they were strained by the "gag," and that they should not be punched in the stem for fish bolts, nor in the feet for spikes. Punching destroys the elasticity about the hole, and establishes the commencement of a fracture. The holes, should, therefore, be drilled, or, better yet, a joint splice should be employed, which does not require holes in the rail.

Tests for Rails.

To insure safety, a variety of tests have been instituted for rails. Upon the Philadelphia, Wilmington, and Baltimore Rail-

road the steel rails are tested for hardness with the chisel, for toughness with a trip-hammer, and for strength with a 2240 pounds drop, falling 15 feet, the rail resting on supports placed three feet apart. The iron rails for the Royal Swedish Railways are placed upon supports 4 feet apart, and a ram of 4480 pounds falls upon the middle of this span from a height of 12 feet. One rail in 100 is thus tested, and if it does not break, the whole 100 are accepted. Should the rail break, the 100 are divided into four lots, and one rail out of each lot is tried. This trial decides whether all or a part of the lots are to be accepted. The rails rejected as of bad quality are immediately cut up by, and at the expense of, the contractor.

The tests for iron rails adopted by the German Railway Union (embracing nearly the whole length of the railroads of Prussia and Northern Germany), are as follows: The rail being placed on solid supports, 3.28 feet apart, is cut $\frac{1}{8}$ inch in the head, and broken bottom up. It is then cut $\frac{1}{8}$ inch in the base, and broken head up. The surface of fracture broken either way must show perfectly granular heads, fibrous flanges, and no open welds. This test determines the quality of the material, and also, to some extent, the workmanship. The drop test lets a ram of half a ton fall 11.48 feet upon the middle of the rail, placed on supports, 3.28 feet apart. This test shows the endurance of the rail against sudden shocks, though it does not show the quality of the material, as the fibrous iron, broken by so sudden a shock, may appear granular. To test the limit of elasticity the rail is placed upon solid supports, 3.28 feet apart, and first $12\frac{1}{2}$ tons are placed on the centre of the rail, which, after 5 minutes' pressure, must not show permanent set; second, 25 tons, applied in the same way for 5 minutes, must not break the rail.

The steel rail tests at the works of John A. Griswold & Co., of Troy, N. Y., are as follows: —

"*First*. A test ingot from each five ton ladleful of liquid steel is hammered into a bar, and tested for malleability and hardness, and especially for toughness, by bending it double, cold. In case

any test bar falls below the standard established as suitable for rails, all the ingots cast from that ladleful of steel are laid aside for other uses.

Second. All the ingots, and each rail rolled from them, are stamped with the number of the charge or ladleful. A piece is cut from one rail in each charge, and tested by placing it on iron supports, a foot apart, and dropping a weight of five tons upon the middle of it, from a height proportioned to the pattern of rail A blow equivalent to a ton weight falling 10 to 15 feet, is considered a severe test. A five ton weight falling from a less height is used upon the ground that it more nearly represents in kind (although it of course exaggerates in severity) the test of actual service in the track.

"In case a test rail does not stand the blow deemed proper and agreed upon, the whole of the rails made from that charge, or ladleful of steel, are marked No. 2, and sold for use in sidings, where their possible breaking would do no great harm, and where their greater hardness and resistance to wear would be specially valuable."

Besides the above tests the rails are rigidly inspected for surface imperfections.*

Steel rails of the best quality may be bent double when cold, or twisted an entire turn in a length of two feet without cracking.

General Remarks upon Rails.

Iron rails, at the best, are never homogeneous, being made up of a large number of separate pieces, which are more or less imperfectly welded together. Such rails do not properly *wear out*.

* The test of rails by the drop must be taken for what it is worth, and no more. Steel rails have been broken in the track into three pieces, when the outside as well as the fractured surfaces appeared entirely free from defects: while the broken pieces could subsequently not be broken under a one ton drop falling 20 feet. It should be remembered that rails in the track, especially when fished, are in constant vibration long before the approaching train passes over them.

They split, or laminate, the top bar splitting off from the body of the rail. To remedy this defect, it has been proposed to arrange the pile in vertical instead of horizontal bars; an arrangement which has produced very good results. The tendency of the rolling of the wheels over the rails is to alter the structure of the upper part of the rail where it is subjected to compression, making it granular. When a rail in this state is moved in the track, so as to bring the strain upon it in a new position, it is found to be considerably damaged. Mr. Vignoles states, that after 7 or 8 years of wear the English double-headed rail, which is made to be reversed when the top is worn out, when turned, snaps off almost immediately.* Mr. Steel observes, that when the head has become granular from the action of the wheels, and its tensile strength destroyed, they not only break when turned, but that any cause which changes the position of the rails in the track, so as to alter the bearing of the wheels, causes their destruction; that if they are reversed side for side to the flanges, they wear out quickly; and even if they are taken up at one point of the road and laid down at another, by which means the bearing of the wheels upon them is of necessity more or less changed, the granulated surface is broken up, and their durability is decreased.†

Rail Joints.

The weakest part of the track is that where, to resist the concussion of the wheels, it should be the strongest, namely, at the joint; for at this point we lose the strength of the rail and depend upon the splice. The old mode of placing the ends of the rails upon a chair, and then pounding rails, chairs, and sleepers to pieces by locomotives, is too unmechanical and too expensive to deserve the slightest notice. If not already entirely abandoned, it should be.

* Discussion on Price Williams' paper on Permanent Way. Proceedings of Inst. Civ. Eng. London, 1866.

† Proceedings of American Society, Civ. Eng. New York, 1870.

The most common practice now is to fish the joint by plates, as shown in Figs. 1 and 2, Plate XXVIII. As generally used, however, the fish bars are very far from making up the strength of the rail. They should not only be deep, but thicker than ordinarily seen. As a girder, resisting by the compression of the top fibres and the extension of the lower ones, the rail is far stronger than the short plates used for splicing. With regard to the strength of the fish joints, Baron Von Weber, in his experiments, placed an 18 feet rail on ties, $3\frac{1}{5}$ feet apart, except the joint ties, which were but 2 feet, when the deflections were the same both at the joint and between the intermediate ties, from the same load ; whence he concludes, that where well-designed, suspended fish joints are used, an almost uniform supporting strength will be given to all parts of the road by making the distance between the joint sleepers equal to 0.6 of that between the intermediates, and by distributing the latter at uniform distances along the length of the rail. The end of the rail is the part which is easiest damaged by blows, the metal in the top surface being less supported by the adjoining material than at any other part. Placing the ends of the rails in a rigid chair, is like placing them on an anvil. If it were desired to destroy the end of the rail in the most rapid manner we should use the old-fashioned unfished joint, and place the ends of the rail in a chair. By suspending the joint between two ties we remove the anvil from beneath the ends, and by strongly fishing the joint we make up, in great part, the lost strength.

A joint which has been received with deserved favor, and designed by Mr. Reeves, is shown in Fig. 3, Plate XXVIII., in which the lower flange is securely clamped, and the joint supported by a short girder reaching between the joint ties. This is particularly applicable to steel rails, as it requires no bolt holes in the rail stem. This joint has been well tested, and has given great satisfaction upon the Philadelphia, Wilmington, and Baltimore Railroad. Additional support and stability may be given to the common fished joint by the short longitudinal timber, represented in the cross section at Fig. 4, Plate XXVIII. To prevent the

longitudinal shoving of the rails, seen especially on grades, termed "creeping," without notching the foot of the rail, a contrivance termed a "stop chair" has been employed upon some roads. This consists simply of a thin piece of plate, stamped into the proper form, the upper flange being screwed in between the nut and fish plate, while the lower flange is spiked directly to the sleeper.

Expansion of Rails.

Wrought iron expands .0000068 of its length for each degree of Fahrenheit. A rail is thus longer in summer than it is in winter. In order that the track may be kept in the right line, both horizontally and vertically, rails laid in cold weather must not be placed in contact, but must be separated by space enough to allow the necessary expansion to take place. This space will depend upon the difference between the temperature of the rail at the time it is laid and the greatest temperature that it will ever reach, and also upon the length of the rail. Iron in the track is frequently found in very hot weather to have a temperature as high as 190 degrees. It is moreover a well-known fact that a bar of iron which has been expanded by heat never quite returns to its original length; so that the bar receives not only a temporary but a permanent elongation. This suggests that even in the hottest weather rails should not be placed quite in contact. The following table gives the allowance to be made for expansion for iron or steel rails of different lengths.

Temperature of the air in degrees Fah.	Allowance in thirty-seconds of an inch, the length of the rail in feet being—					
	16	20	24	28	32	36
90°	2	3	4	5	6	7
80°	3	4	5	6	7	8
70°	4	5	6	7	8	9
60°	5	6	7	8	9	10
50°	6	7	8	9	10	11
40°	7	8	9	10	11	12
30°	8	9	10	11	12	13
20°	9	10	11	12	13	14

For neglecting this simple precaution, and by placing rails too close in cold weather, many serious and fatal disasters have occurred, for which there is certainly no excuse. Track laid close in cold weather has been thrown out of line and out of level more than a foot; and this occurs, however heavy and well fastened the track may be. A train upon the North Eastern Railway, England, in June, 1856, was thrown off the inside of a curve while running at 40 miles per hour, on account of the lack of provision for expansion in the rails, notwithstanding that the latter weighed 82 pounds per yard, and were fished at the joints, and very securely fastened to the chairs. Any rail or any fastening is weak compared with this powerful expansive force. In fishing the joints, the holes in the rails must, of course, be longer than the bolt, to allow the proper motion from expansion.

Elevation of Exterior Rail.

The motion of a train of cars around a curve is accompanied by a tangential force, depending in amount upon the velocity of the train and the radius of curvature. This force tends to throw the cars against the outer rail, a tendency which is resisted by elevating it to a certain height above the inner one. The amount of this elevation is found by the rule, —

$$\frac{V^2 g}{32\ R} = E.$$

In which V is the velocity in feet per second;
g, the gauge of the road in inches;
R, the radius of curvature in feet, and
E, the elevation of the outer rail in inches.

Thus the speed being 30 miles an hour, the gauge 4′ 8½″, or 4.7 feet, and the radius of curvature 5730 feet, the elevation in inches becomes, —

$$\frac{44^2 \times 4.7 \times 12}{32 \times 5730} = 0.6 \text{ inches}.$$

In this way the following table is computed :—

Degree of Curvature.	Radius of Curvature.	Elevation of the outer Rail in Inches, the Velocity in Miles per Hour being, 10	15	20	25	30
1	5730	.07	.15	.26	.41	.59
2	2865	.13	.30	.53	.82	1.18
3	1910	.20	.45	.79	1.24	1.78
4	1433	.26	.60	1.06	1.65	2.38
5	1146	.33	.74	1.32	2.07	2.97
6	955	.39	.89	1.59	2.48	3.56
7	819	.46	1.04	1.85	2.89	4.16
8	717	.53	1.19	2.11	3.30	4.76
9	637	.59	1.34	2.38	3.72	5.36
10	574	.66	1.49	2.64	4.13	5.94

The elevation is given to the outer rail, when the track is first laid, by means of the levels correctly determined by the engineer; but for the subsequent maintaining of the elevation no provision is made except the eye of the trackman. A spirit level, and a small arc, graduated according to the elevation required for different curves, may be attached to the rail gauge, and the elevation being fixed for each curve upon the road for the highest speed, the proper elevation will be kept.

Coning of Wheels and Widening of the Gauge.

Besides the tendency to run against the outer rail from the centrifugal, or rather tangential force, the parallelism of the axles and the difference in length between the outer and inner rails have the effect of forcing the wheels against the outer rail. The effect of the difference in length between the inner and outer

rails is to some extent neutralized by making the tread of the wheel conical instead of cylindrical, by which the diameter of the outer wheel is increased while that of the inner one is reduced, thus making the respective diameters proportioned to the distances the wheels have to traverse. The coning of the wheel has, however, the effect of causing the cars to oscillate laterally to some extent when upon straight lines. The amount of cone will be noticed in the remarks in advance, in Chapter XVIII.

Upon sharp curves it is customary to widen the gauge from half an inch on radii from 750 to 1000 feet, to $\frac{3}{4}$ of an inch for a radius of 500 feet, and an inch for curves of less radius. Upon the Prussian railways the gauge upon tangents and curves of over 1000 feet radius is 4′ 8½″, and upon sharp curves 4′ 9½″.

Crossings, Switches, Frogs.

In crossing from one track to another the rails are arranged as in Fig. 153. This is the single crossing. Fig. 154 shows the

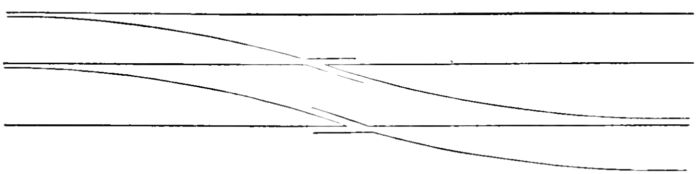


Fig. 153.

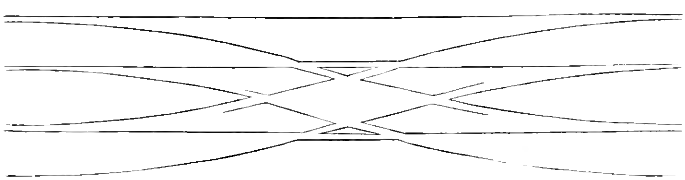


Fig. 154.

arrangement for a double crossing, and Fig. 155 the connection between a main track and two side tracks. The switch consists of one length of rails made movable at one end, and so attached to a suitable lever that they may connect with either line of

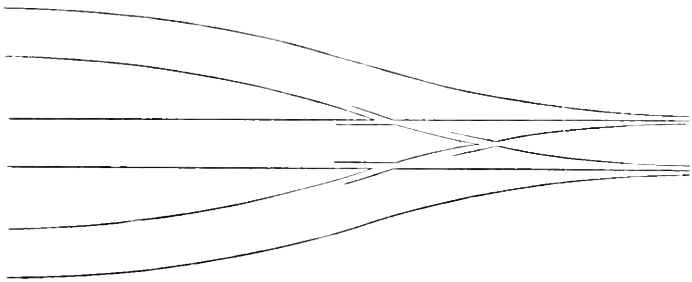


Fig. 155.

rails upon which a train is to be run. The simple switch leaves one line of rails always open, while the other line is continuous. To avoid this defect, by which numerous disasters have occurred, a variety of plans have been designed. The Tyler Switch consists of a long set of castings, so arranged that a pair of wheels approaching upon the broken track are run up on to the casting, crowded over sidewise, and finally dropped down upon the continuous line. The Wharton Switch leaves both rails of the main track at all times unbroken and undisturbed, so that entire security is given to trains traversing both the main and the side track, even if the switch is left wrong. The above, as well as a variety of other arrangements to effect the same end, are understood much easier by a few moments' inspection than by any description. Where one line of rails crosses another the passage of the wheels is effected by means of a frog, several forms of which are shown in Fig. 5, Plate XXVIII. That the wheel may run smoothly from A to B, the rail C D must be cut at E; and that the wheel may run smoothly from C to D, the rail A B must be cut at the same point. Cutting the two rails gives the general form shown in the figure. In order that the flange of the wheel shall not leave the line A B when it is at the break E, the guard rail M M is used to confine the opposite wheel for a short distance. It should be placed parallel with, and two inches from the outer rail F F, and should reach a short distance beyond the opposite opening in each direction. At the ends the guard rail should curve gently away,

as in the figure; the wheel will thus be brought gradually into the right line, and kept so until the break in the opposite rail is passed, and will afterwards be easily released. The guard rail may be braced by struts reaching across the track and fastened down, or both the main and guard rails may be put into a double chair, fastened to the sleepers.

The dimensions of the frog will depend upon the angle at which the rails cross each other, and upon the size and form of the wheel flange. The form and size of the standard tires, as made by the Portland Company, is shown in Plate XXVIII., Figs. 9 and 10, the first being for cars and the second for engines. The width of the channel for the passage of the flange through the frog is made from $1\frac{3}{4}$ to 2 inches. The old cast iron frog had steel points and shoulders, the ends of the casting being spread out into a broad chair, to receive the ends of the rails and to secure the frog to the ties. Additional flanges for fastening the frog at one or more intermediate points between the ends, according to the length of the casting, were also provided. The body of this frog was of cast iron, the point of solid steel, and the remainder of the point and the sides were covered with a thick plate of steel. The edges of the point were rounded off in the same manner as the head of the rail, in order to fit the shape of the tire. The above frog was placed upon large and well-bedded cross ties, and firmly spiked in order to resist the extra shocks occurring at the crossing.

A better method of making a frog is that shown in Fig. 5, Plate XXVIII., where the rails themselves are cut and bent, the whole being supported upon a broad and firm wooden bearing. In this case the rails should be of steel, or, if of iron, a steel point and steel tops should be put on to the rails. It should be borne in mind that frogs and crossings are subject to a greater wear than any other part of the track, as they are employed for turnouts from the main line, and thus involve sharp curves. Besides, the work is very severe upon side tracks, where the engines are continually employed in making up trains. The tracks at such places should

therefore be laid with especial care upon a solid but elastic bed, and well drained by an ample supply of good ballast.

Several frogs have been designed with a view of securing a certain degree of elasticity at the crossing, which is no less desirable here than at other points of the track. The Mansfield frog, shown in plan by Fig. 6, and also represented in section by Figs. 7 and 8, consists of a point and rails of steel, resting upon a wrought iron plate. This plate bears upon an oak plank, under which is another wrought iron plate followed by a thin sheet of pure rubber, and beneath this a second plank of oak resting upon a bottom plate of wrought iron. This arrangement combines hardness upon the surface with strength and elasticity below. The cost of such a frog is about double that of the common cast iron one, while its durability is very much greater, and its effect upon the rolling stock much less damaging. Other combinations of steel, wrought iron, and rubber have been made, and have given excellent results.

A cast steel frog, a section of which is represented in Fig. 12, has been made at the Butcher Steel Works in Philadelphia, the whole being in a single piece, and made, with a double working face, so as to be reversible.

"Crossings" occur where two tracks intersect, and consist of four frogs and the corresponding guard rails, as shown in Fig. 5, and in Fig. 11, the whole being placed upon a substantial and firmly connected bed of timbers.

Each rail in a curved track requires to be bent to fit the curve before being laid. This is done by placing the rail upon supports at the proper distance, and applying pressure to it by means of a lever or a screw.

For ample discussions of the numerous problems relating to the bending of rails, the putting in of frogs and switches, and the running of the various curves required in railway work, with tables for facilitating the practical operations, the reader is referred to the works of Trautwine and of Henck.

CHAPTER XVII.

THE LOCOMOTIVE ENGINE.

As the locomotive engine is the power by which railroads are worked, and as its proportions and dimensions are so intimately connected with the physical character of the road, it is thought proper to take space enough at this point to examine the general principles of its construction, and of its adaptation to the work required of it upon railroads.

The locomotive is a non-condensing, high-pressure engine, working at a greater or less degree of expansion, according to the labor to be performed, and placed upon wheels, which are so connected with the piston that any motion of the latter is communicated to the former, by which the whole is moved.

The power exerted in the cylinder, and referred to the circumference of the driving wheel, is called *traction;* its amount depends upon the cylinder diameter and steam pressure, and upon the diameter of wheel and the stroke, this latter being the distance traversed by the piston from one end of the cylinder to the other.

The means by which the "traction" is rendered available for moving the engine and its load is the resistance which the wheel offers to slipping on the rail, or its bite, and is called *adhesion;* it is directly as the weight applied to the wheels, but depends also upon the state of the rails. It varies from nothing, when there is ice on the rail, to one fifth of the weight upon the driving wheels when the rail is clean and dry, and in some cases has reached as high as nearly one third.

Steam-producing, Traction, and *Adhesion,* are the three ele-

ments which determine the ability of an engine to perform work. The proportions and dimensions of the machine depend upon the duty required of it; sufficient adhesion for a required effect should be obtained rather by a proper *distribution,* than by increase of weight.

The work required of any engine depends upon the nature and amount of traffic, and upon the physical character of the road.

The nature of the traffic, whether bulky or compact, and whether requiring quick or slow transport, determines somewhat the number and size of the trains, and, consequently, the number and power of the engines.

A road with steep grades and sharp curves, with the same amount of traffic, will need stronger engines than a road with easy grades and large curves.

The amount of motive power and cost of working it depends in a great degree upon the disposition of grades as regards the direction of the traffic movement. The most economically worked road will be either a level one, or one where the bulk of the traffic is moved down hill.

The mineral, commercial, or agricultural nature of the country determines the direction of the traffic, and the physical nature the arrangement of the grades.

The different kinds of labor required of locomotives necessitate the employment of engines of different proportions; and the different classes of railways require engines possessing different amounts of power.

High rates of speed are generally combined with light loads, and heavy trains are required to move at the lower velocities.

Great speeds require the rapid production and consumption of a large bulk of steam of but little density, large wheels and short stroke, that the ratio of velocities of piston and wheel may be as great as possible.

Heavy trains require steam of a much greater density, and combine a long stroke with a small wheel, by which great leverage is obtained.

In general, engines for winter use should be heavier than those for summer, upon the same ground, as natural causes are more liable to resist adhesion in the winter.

The locomotive engine may be so proportioned as to run at any speed, from 10 to 60 miles an hour: it may ascend grades from 20 to 200 feet per mile; and haul from 100 to 1000 tons.

The rules by which the necessary dimensions to perform any required duty are fixed, depend upon the very simplest mechanical laws. The ruling grades, and the weight and speed of trains, determine the amount of power required of the engine, while the ability to supply the power is dependent upon the capacity of the engine for producing steam, the leverage by which that steam is applied, i. e., the ratio between the stroke and wheel diameter, and the weight upon the driving wheels, by which the adhesion is controlled.

Resistances to Motion.

The exact resistance to the motion of a railroad train cannot be determined, as some of the elements are so variable, for example, the state of the weather. An approximate estimate, near enough for practice, is easily obtained. To arrive at correct data, the observations must be made upon trains working under the ordinary conditions to which they are subject in practice.

The whole resistance is made up of several partial resistances, some of which are constant at all speeds, and some of which increase with the velocity.

The engine and tender resistance is composed of the friction of pistons, cross heads, slide valves, cranks, eccentrics, pumps, the back pressure of the blast, and various erratic movements, rolling, twisting, and pitching, together with both wheel and axle friction, which is common to the engine and tender.

The train resistance arises from the displacement of the atmosphere, from oscillation of the cars, and from the friction of the wheels and axles.

The atmospheric resistance is due, not only to the direct action

of the air upon the front and sides of the train, but also to the exhausting action in the rear. It thus depends largely upon the bulk of air displaced, and upon the time occupied in displacing it, and thus upon the bulk and speed of the train. The train has, as it were, to pull along a large column of air, like the water in the wake of a ship: form or amount of frontage has little or no effect. A train with the same frontage offers more resistance as its bulk increases.

Oscillatory resistance is caused by irregularities in the surface of the rails, and increases with the velocity, and also with increase of height of the centre of gravity of the car or engine.

Frictional resistance may be divided into wheel and axle friction. That of the axle is composed of two parts, the direct vertical friction on the journal, and the side friction on the collars, consequent upon lateral motion. The direct cause of the vertical friction is the weight of the car, and that of the lateral friction the irregularities in the surface of the rails, which cause the car to sway from side to side. As the diameter of wheel increases the oscillation is increased, the centre of gravity being raised. Wheel friction, which acts between the periphery of the wheel and the surface of the rail, increases with the load, and, to a small extent, depends upon the wheel diameter.

For the total resistance to the movement of a railway train Mr. D. K. Clark gives the following rule, where R is the resistance in pounds per ton of engine, tender, and train, and V the velocity in miles per hour.*

$$\frac{V^2}{171} + 8 = R.$$

The rules given by Vuillemin, Guebhard, and Dieudonné furnish results very nearly the same as the above, being a little less for low velocities, and a little more for high speeds, but being almost identical for velocities from 20 to 40 miles an hour.†

* Railway Machinery, Part IV., Chapter II., p. 298.

† *De la Résistance des Trains et de la Puissance des Machines.* Paris, 1868.

From Mr. Clark's rule, the following table is constructed, showing the resistance in pounds per ton for different velocities, and also the total resistance in pounds for different loads at different speeds:—

Velocity in Miles per hour.	Resistance in Pounds per Ton.	Total Resistance in Pounds for a Gross Load of— 50 Tons.	75 Tons.	100 Tons.	250 Tons.	500 Tons.	750 Tons.	1000 Tons.
10	8.6	429	644	858	2146	4292	6439	8585
12	8.8	442	663	884	2211	4421	6632	8842
15	9.3	466	699	932	2329	4658	6987	9315
20	10.3	517	775	1034	2585	5170	7754	10340
25	11.7	583	874	1165	2913	5827	8740	11654
30	13.3	663	995	1326	3316	6632	9950	13264

From a great number of experiments, Mr. Clark concludes that the relative resistance to the motion of inside and outside connected engines is as 17 to 14. The effect of other conditions according to the same authority, is to add the following percentages to the resistance:—

Bad state of the road, ...	40
Curves, ...	20
Strong head and side winds, ...	20
In all, ...	80

The resistance per ton is greater with light or empty trains than with those that are well loaded, since the less the useful load the greater the number of wheels and axles for the same gross tonnage. Curves of large radius, say Vuillemin, Guebhard, and Dieudonné, have no effect; but with a radius as small as 1000 metres (3820 feet) an increase of length produces an increase of resistance: thus, between Paris and Strasbourg, at a point where curves of 1000 metres radius are frequent, trains of 35 to 50 wagons demand on an average 2.2 pounds per ton more than a train of 25 to 30 wagons; with trains of from 10 to 20 passenger

cars (French), and a speed from 20 to 30 miles an hour, curves of 2624 feet radius are not felt; but above 30 miles an hour the resistance makes itself sensible.*

The resistance due to any grade, as already remarked in a preceding chapter, is found by multiplying the load by the rise, and dividing by the length of the incline. The following table gives the resistances for loads from 1 to 1000 tons upon grades of from 10 to 100 feet per mile, the numbers being in pounds: —

Grade.	1	50.	75.	100.	250.	500	750.	1000
10	4	212	318	424	1,061	2,121	3,182	4,242
20	8	424	636	848	2,121	4,242	6,364	8,484
30	13	636	955	1,273	3,182	6,363	9,545	12,726
40	17	848	1,273	1,697	4,242	8,484	12,727	16,969
50	21	1,061	1,591	2,121	5,303	10,606	15,909	21,212
60	25	1,273	1,909	2,545	6,364	12,727	19,091	25,454
70	30	1,485	2,227	2,970	7,424	14,848	22,273	29,696
80	34	1,697	2,545	3,394	8,485	16,969	25,454	33,939
100	42	2,121	3,182	4,242	10,606	21,212	31,818	42,424

TRACTIVE POWER.

The whole steam pressure upon both pistons, referred by means of the connecting rod, crank, and wheel to the rail, is called *traction*. It is the drawing power of the engine. Its amount depends upon the diameter of cylinder, steam pressure, stroke, and diameter of wheel.

By increasing the steam pressure, we increase the power. By increasing the cylinder diameter, we increase the power. By increasing the stroke, we increase the power. By decreasing the wheel diameter, we increase the power. And by adjusting the

* See Appendix. Experiments of B. H. Latrobe, Esq., on Baltimore and Ohio Railroad, and experimental trip upon the New York and Erie Railroad.

dimensions of the above parts, we may give any desired amount of power to the engine.

The formula expressing the tractive power of an engine, of any dimensions, is

$$\frac{(2\ A)\ P \times 2\ S}{C}.$$

Where A = the area of one piston ;
P = the pressure in pounds per sq. inch on the piston ;
S = the stroke in inches ;
C = the circumference of the wheel in inches.

In other words, the total pressure on both pistons multiplied by the ratio between the double stroke and the wheel circumference gives the tractive power of the engine in pounds. The piston pressures for different unit pressures, and for different cylinders, are given in the following table :—

Diam. of Cylinder.	Area of Piston.	Whole Pressure on both Pistons at a unit Pressure in Pounds per sq Inch of					
		50.	60.	70.	80.	90.	100.
12	113.1	11,310	13,572	15,834	18,096	20,358	22,620
13	132.7	13,270	15,924	18,578	21,232	23,886	26,540
14	153.9	15,390	18,468	21,546	24,624	27,702	30,780
15	176.7	17,670	21,204	24,738	28,272	31,806	35,340
16	201.1	20,110	24,132	28,154	32,176	36,198	40,220
17	227.0	22,700	27,240	31,780	36,320	40,860	45,400
18	254.5	25,450	30,540	35,630	40,720	45,810	50,900
19	283.5	28,350	34,020	39,690	45,360	51,030	56,700
20	314.2	31,420	37,704	43,988	50,272	56,556	62,840
21	346.4	34,640	41,568	48,496	55,424	62,352	69,280
22	380.1	38,010	45,612	53,214	60,816	68,418	76,020
23	415.5	41,550	49,860	58,170	66,480	74,790	83,100
24	452.4	45,240	54,288	63,336	72,384	81,432	90,480

The ratio between the double stroke and wheel circumference is given in the table below: —

Wheel. Diameter in Feet.	Ratio between Double Stroke and Circumference of Wheel, which, multiplied by the total Pressure on both Pistons, gives the Tractive Power of the Engine in Pounds, the length of Stroke being — 18 Inches.	20 Inches.	22 Inches.	24 Inches.
$3\frac{1}{2}$	.2728	.3031	.3334	.3637
$3\frac{3}{4}$	.2546	.2829	.3112	.3395
4	.2387	.2652	.2917	.3183
$4\frac{1}{4}$	.2246	.2496	.2746	.2996
$4\frac{1}{2}$	.2122	.2357	.2593	.2829
$4\frac{3}{4}$	.2010	.2233	.2457	.2680
5	.1909	.2122	.2334	.2546
$5\frac{1}{4}$	.1818	.2021	.2223	.2425
$5\frac{1}{2}$	.1736	.1929	.2122	.2315
$5\frac{3}{4}$	.1660	.1845	.2029	.2214
6	.1591	1768	.1945	.2122

Thus the tractive power with a mean effective pressure upon the pistons of 80 pounds per inch, the stroke being 22 inches, and the cylinder diameter 16 inches, with a wheel 5 feet, is —

By table of piston pressures, ...	32,176
Ratio in the table above, ...	.2334
Tractive power of the engine in pounds, .	7509.8

ADHESION.

As before remarked, the adhesion, or the bite of the wheels upon the rails, varies from a fifth to an eighth of the weight upon the drivers; so that to obtain a tractive power of 5000 pounds we require a weight of from 25,000 to 40,000 pounds upon the driving

wheels. With a clean, dry rail in summer, an adhesion of one fifth the weight upon the drivers will often be obtained; in ordinary practice one seventh is probably a fair amount, while in wet and snowy weather it will fall below even a tenth.*

FUEL.

The fuel employed in the locomotive is either wood, coal, or coke, though the latter is rarely used in this country. The relative economy of the several fuels depends upon their cost, and upon their power to produce heat. The cost is governed by locality and by the value of labor, while the heat-producing power depends upon the chemical constitution. The most economical fuel is to be determined for each road, and even upon the same line coal may be cheaper upon one part and wood upon another. Experiments made upon different fuels show the evaporating power of a ton (2240 pounds) of Cumberland coal to be equal to 1.25 tons of anthracite, or 1.75 cords of pine wood. A pound of good coal evaporates 9 pounds of water, while a pound of the best wood evaporates 4.5 pounds of water, and a pound of coke about the same as coal. On account of bad seasoning and other causes, a cord of average wood cannot be reckoned as equal to more than a half ton of good bituminous coal. The consumption of fuel in actual practice upon railways is stated by Mr. Colburn † to have been in England,

* In the trial trip made upon the New York and Erie Railway in 1855, an adhesion of one third the weight upon the drivers was frequently attained: while the resistance to the train was unusually low, being only from 6 to 7 pounds per ton, the speed being 5 miles an hour. So great an adhesion, however, is rare, and not to be counted upon except under the most favorable conditions of track and weather. Mr. Latrobe, in his discussion of the location of the Portland and Ogdensburgh Railway, through the White Mountain Notch, considers an adhesion of one seventh all that can be relied upon, and the same fraction is taken by Professor Rankine, in his Manual of Civil Engineering. Vuillemin, Guebhard, and Dieudonné give the adhesion as from one fifth to one ninth. — *Résistance des Trains*, etc., p. 64.

† Permanent Way and Coal Burning Locomotive Boilers of European Railways. 1858.

from 1850 to 1856, from 20 pounds of coke per mile, or 100 miles per ton, to 50 pounds per mile, or 40 miles per ton; varying, of course, with the weight of the trains. The same authority gives the average run per cord of wood in the Northern States of America as about 25 miles. Upon the Baltimore and Ohio Road the average of all trains in 1857 was 33.5 miles run per ton of bituminous coal, or 67 pounds per mile. Upon the Pennsylvania Railroad the general average, in 1868, was 64.8 pounds per mile run; in 1869, 62.9 pounds per mile; or about 35 miles per ton. Upon the Illinois Central Railway the miles run per ton of coal vary from 30 to 50, or at the rate of from 40 to 70 pounds per mile. Knowing the market price and the evaporative efficiency of the different fuels at any particular place, we see at once which is to be preferred; or knowing the price of one variety, we find at once the limit to be paid for the other. Thus wood, being valued as in column 1 of the table below, we may pay for coal the prices in the remaining columns, as the comparative efficiency has different values:—

Value of a Cord of Wood being.	Price that may be paid for a Ton of Coal when		
	1 Ton = $1\frac{1}{2}$ Cords.	**1 Ton = $1\frac{3}{4}$ Cords.**	**1 Ton = 2 Cords.**
$3.00	$4.50	$5.25	$6.00
4.00	6.00	7.00	8.00
5.00	7.50	8.75	10.00
6.00	9.00	10.50	12.00

In ordinary practice the fuel is not consumed with the full economy that should be obtained in the locomotive. The short time allowed for combustion, the small size of the furnace, and withal the lack of care upon the part of the person in charge of the engine, render the expense of fuel much larger than it should be. An occasional experiment will develop the fact that judi-

cious management of the fire, and close attention to running the engine, will increase the effect of a ton of coal even from 25 to 50 per cent. There are several reasons why all the heat which the fuel may furnish is not obtained. From a lack of a sufficient supply of air the inflammable gases evolved by the heat are not all consumed; that drawn through the fire by the draught being only sufficient to decompose more fuel than when decomposed it can burn. The thick smoke that escapes from a chimney when fresh fuel is thrown upon a hot fire is unconsumed gas, decomposed fuel, but without air enough to burn, although the supply of heat may have been ample. All of the so-called "smoke consuming" furnaces, are simply furnaces in which means are provided for admitting fresh air to the unconsumed gases above the fire, and for obtaining *time* for combustion. This air, however, must be admitted while the gases have heat enough to burn, i. e., it must be done before the smoke enters the flues. Enlarging the furnace and admitting the air above the fire, either by hollow stay bolts or by small holes in the fire door, completes the combustion; the large grate area and the roomy furnace affording the great essential *time* for the fuel to burn.*

Generation of Steam.

The heat generated in the fire-box is conducted through the tubes to the exhaust chamber, during which passage it is imparted to the metal, and from thence absorbed by the adjacent water, which being thereby made lighter, rises to the surface and gives place to a new supply. The duty of the furnace is to *generate*, and of the tubes to *communicate*, heat.

* The reader is referred to the work of Messrs. Colburn and Holley upon Coal Burning Boilers, already mentioned, for numerous illustrations of locomotive furnaces designed for the more perfect combustion of fuel. Very few of the plans proposed, however, have received the approval of railway managers. The best coal burning engines have simply roomy furnaces, with a combustion chamber in the back part of the barrel, the furnaces in many freight engines being no less than 10 feet in length.

The tubes, or flues, varying in number from 100 to 300, in diameter from 1½ to 2½ inches, and in length from 8 to 16 feet, furnish the real communicating surface. The amount of heating surface thus obtained for any length, number, and diameter, is given in the following table: —

Length of Tube in Feet.	Square Feet of Surface given by one Tube, the Diameter in Inches being —							
	1½	1¾	1⅞	2	2⅛	2¼	2⅜	2½
8	3.14	3.66	3.93	4.19	4.45	4.71	4.97	5.23
8½	3.34	3.89	4.17	4.45	4.73	5.00	5.28	5.56
9	3.53	4.12	4.42	4.71	5.00	5.30	5.59	5.88
9½	3.73	4.35	4.66	4.97	5.29	5.59	5.90	6.21
10	3.93	4.57	4.91	5.24	5.56	5.89	6.21	6.54
10½	4.12	4.80	5.15	5.49	5.84	6.18	6.52	6.87
11	4.32	5.03	5.40	5.76	6.11	6.47	6.84	7.20
11½	4.51	5.25	5.64	6.02	6.39	6.77	7.15	7.53
12	4.71	5.49	5.89	6.28	6.67	7.06	7.46	7.86
12½	4.91	5.72	6.14	6.54	6.94	7.36	7.77	8.18
13	5.10	5.95	6.38	6.80	7.23	7.65	8.08	8.51
13½	5.30	6.18	6.63	7.07	7.50	7.94	8.39	8.84
14	5.49	6.40	6.87	7.33	7.78	8.24	8.70	9.16
14½	5.69	6.63	7.12	7.59	8.06	8.53	9.01	9.49
15	5.89	6.86	7.36	7.85	8.34	8.83	9.32	9.82
15½	6.08	7.09	7.61	8.11	8.61	9.12	9.63	10.15
16	6.28	7.32	7.85	8.37	8.89	9.41	9.95	10.48

The power of a plain surface to generate steam depends upon its position, and on the ability of the material to transmit heat.

An experiment recorded in Clark's Railway Machinery gave the following results: A cubic metallic box submerged in water, and heated from within, generated steam from its upper surface more than twice as fast as from the sides when vertical, while the bottom yielded none at all. By slightly inclining the box, the elevated side produced steam much faster, while the depressed one parted so badly with it as to cause over-heating of the metal. Acting upon this result, most builders of engines of the present day give an inclination of from one inch to one quarter of an inch per foot to the sides of the inner fire-box. That the heat should be applied in the most effectual manner to the water, the latter should *circulate* freely around the hot metal, carrying off the heat as soon as it reaches the surface.

The efficiency of circular tubes is a matter not yet fully understood. They certainly give a large amount of surface in a small boiler. Pambour considered the value of tube area, per unit of surface, in terms of the furnace area, as one third only; that is, three square feet of tube surface, as equal to one foot of furnace area, in power of generating steam. Mr. D. K. Clark makes no distinction between the two surfaces, but observes, "There is reason to believe that in the upper semicircular part of each tube the efficiency principally resides. The winding progressive motion observable in tubes of considerable diameter confirms this conclusion, as it is, with much probability, due to the cooling of the upper portion of the gases of combustion, which, as they cool, also become heavier, and descend laterally to make room for the hotter smoke next the bottom of the flue, the general result of which is the spiral motion of the current in its progress onwards." Certainly the upper half of the tube would part much easier with the steam than the under one, even supposing the applied heat to be the same.

The nature and amount of work to be done by the locomotive determines the quantity and quality of steam to be produced. The following table shows the properties of saturated steam:—

Steam Pressure Pounds per Inch.	Relative Volume, or Cubic Feet of Steam, Water being 1.	Temperature in Degrees Fahrenheit	Total Heat in Degrees, Fahrenheit.	Weight of a Cubic Foot in Pounds.
50	518	281	1200	.1202
60	437	293	1203	.1425
65	405	298	1205	.1538
70	378	303	1206	.1648
75	353	307	1208	.1759
80	333	312	1209	.1869
90	298	320	1212	.2089
100	270	328	1214	.2307
110	247	335	1216	.2521
120	227	341	1218	.2759
130	211	347	1220	.2977
140	197	353	1221	.3184
150	184	358	1223	.3397

Steam produced in contact with water, or saturated steam, is at its maximum density and pressure for its temperature. If heat be withdrawn from such steam, a part of it will be precipitated as water, and the pressure and density will fall. If, on the other hand, an isolated volume of saturated steam be heated, the temperature and pressure is raised, the volume and density being constant. Saturation is then no more, and the steam is termed surcharged. Mr. Clark, in his fine work upon Railway Machinery, urges the importance of thoroughly drying the steam before applying it to the pistons, as a large gain may thus be made in the effective pressure, especially at high velocities.

Application of Steam.

The steam being generated in the boiler and conveyed to the cylinders is admitted alternately to the opposite sides of the piston, by which its reciprocations are produced. The first valve applied to regulating the admission of steam to the cylinder was so arranged that the steam was admitted during the whole stroke, at the end of which, ingress stopped, and egress commenced at the first end, and ingress commenced at the second end, simultaneously. This caused an unnecessary resistance to the return movement, by preventing the quick escape of the first cylinderful, which had to be *pushed* out instead of *flowing* out. The continuance of the full pressure upon the piston, until the end of the stroke, also caused a dangerous momentum to be given to the reciprocating machinery. These evils are obviated by causing the exhaust passage to open, and the entering port to close, somewhat *before* the end of the stroke.

It is well ascertained, that with very free steam entrances, if we allow the cylinder to be only partially filled, and then cause the steam to expand, more work is accomplished with a given bulk than when the cylinder is completely filled. That the steam may have time thus to expand, the return of the piston must not take place until after the suppression (the stopping of admission).

There are several positions of the valve during each stroke, and several distinct actions of the steam during the same period. The first movement of the valve is to open the steam port, and to admit steam to the cylinder, when the piston commences to move to the opposite end. The next movement of the valve is to close the steam port, and to stop the admission of the steam, which then expands until the stroke of the piston is nearly finished, when the valve, by its movement, opens the exhaust passage, and also at the same time admits steam to the other side of the piston, which, returning, drives out the first cylinderful of steam into the blast pipe, and thence up the chimney. The first cylinderful

of steam, upon the return of the piston, is compressed to some extent, being unable to escape all at once. We have thus the three actions of entrance, expansion, and compression of the steam; and the duration of each of these actions is regulated by the proportions of the valve, and of the valve gear, which may be varied so as to fix the suppression and release of the steam at any desired point. The longer the time between the suppression and release, the more complete will be the expansion.

By the simple connection between the valve and the eccentrics at first employed, any desired rate of expansion was established, but when once fixed, remained the same. To vary the rate of expansion the link-motion has been applied, by which the travel of the valve is made greater or less, thus cutting the steam off at any desired part of the stroke, and regulating the force applied to the piston according to the work to be done, at any time, under whatever conditions the engine may be working.

If we cut the steam off at half stroke, and then allow it to expand, of course the mean pressure during the whole stroke is less than that at entering. The effective mean pressure obtained by any degree of expansion is shown by the following formula, as given by Mr. D. K. Clark in his Railway Machinery.

$$P = 13.5 \sqrt{a} - 28.$$

In the formula, P is the effective mean pressure, in hundredths of the maximum pressure; and a the percentage of admission, in hundredths of the stroke.

Mr. Clark deduces, as general results, from a very extensive and carefully conducted system of experiments, the following:—

That the maximum useful admission is seventy-five per cent., and the minimum, ten per cent. That the greatest possible gain by working expansively is one hundred per cent., which is effected by an admission of ten per cent. That the best admission for engines having ports $\frac{1}{14}$ of the area of the piston, and a blast area from $\frac{1}{13}$ to $\frac{1}{16}$ of the area of the piston, at high speeds (from thirty to sixty miles per hour) and with considerable loads, is from sixty to

sixty-six per cent. With a wider port and blast area, the best admission is seventy-five per cent.

From the formula on page 390 the following table is made:—

Initial Pressure in lbs.	Mean Cylinder Pressure, Admission being in Hundredths of the Stroke. 10	15	20	25	30	35	40	45	50	55	60	65	70	75
50	7	12	16	20	23	26	28	31	33	36	38	40	42	44
60	9	14	19	24	28	31	34	37	40	43	46	49	51	53
70	10	17	22	28	33	36	40	43	47	50	54	57	59	62
80	12	19	26	32	38	42	41	49	54	58	62	65	68	71
90	13	22	29	36	42	47	51	54	60	65	69	73	76	80
100	15	24	32	40	47	52	57	62	67	72	77	81	85	89
110	16	26	35	44	52	57	63	68	74	79	85	89	93	98
120	18	29	38	48	56	62	68	74	80	86	91	97	102	107
130	19	31	42	52	61	68	74	81	87	94	99	105	110	116
145	21	34	45	56	65	73	80	87	94	101	107	113	119	125
160	22	36	48	60	70	78	85	93	100	108	114	121	127	134

Proportions of Parts.

That an engine may perform its work in the most economical manner, a certain proportion should exist between the steam-producing and the steam-consuming parts.

The result of some sixty experiments upon forty-five different engines (detailed in Mr. Clark's work), gives the following formula, expressing the relations which ought to exist between the grate area, the heating surface, and the consumption of water, that evaporation may be carried on in the most economical manner:—

$$S = \sqrt{ac} \times 21.2.$$

Where S = the heating surface in square feet ;
a = the grate area in square feet ;
c = the hourly consumption of water in cubic feet.

The values of a and c in the above formula become —

$$a = \frac{\left(\frac{S}{21.2}\right)^2}{c} \qquad c = \frac{\left(\frac{S}{21.2}\right)^2}{a}.$$

The maximum evaporation which should be carried on per square foot of grate is found by Mr. Clark to be 16 cubic feet of water per hour. Thus, if we wish to evaporate 160 cubic feet of water per hour, we must have a grate area of at least ten square feet.

The smoke box is the general termination of the flues, and the place where the vacuum is produced, which causes the draught. The size of the boiler being the same, the vacuum varies directly as the blast pressure. The power of the blast is, of course, affected by the capacity of the smoke box, which Mr. Clark fixes at three cubic feet per square foot of grate. The vacuum in the furnace varies from one to two thirds of that in the smoke box. The less the resistance to the hot gases experienced in the flues the less may be the vacuum. Upon the vacuum depends the amount of air drawn through the grate, upon the bulk of air drawn through the grate depends the combustion, upon the combustion the evaporation. Whence the evaporation, other things being equal, depends upon the vacuum in the smoke box.

The blast pipe conducts the waste steam from the cylinder, which drives the air from the chimney, and produces the vacuum in the smoke box. Its form should permit the freest escape of the steam. The area of the blast pipe should nowhere be smaller than the exit port, except at the contraction at the top. Too much care, says Mr. Clark, cannot be taken to adjust the blast pipe concentrically with the chimney ; one half inch of eccentricity has been known to damage materially the draught of an

engine. The area of orifice is one of the most critical and most important items in the composition of the locomotive. Increase of the grate area, increase of the tube area, decrease of the length of the tubes, or decrease of the capacity of the smoke box, will allow of a milder blast.

The following proportions are collected from the work of Mr. Clark. The names of the different parts of the engine are arranged below in the order of their importance with relation to the blast. The figures give the best ratios which may be had under the most favorable circumstances :—

Grate area,	1
Ferrule area (area of section of tubes at back flue sheet),	$\frac{1}{5}$
Tube area,	$\frac{1}{4}$
Capacity of smoke box, cubic feet,	3
Chimney (height, four diameters), area of section,	$\frac{1}{15}$
Blast orifice,	$\frac{1}{75}$

The vacuum in the smoke box may, if desired, be regulated by a damper placed in front of the ash pan, by an adjustable nozzle to the blast pipe, or by a blind covering the front ends of the tubes.

Frame and Wheels.

The frame of the engine is the base to which everything should be attached. The cylinders and the wheels both being connected by the frame, it, of course, becomes the counterpart to the piston and connecting rod, and its strength must be able to resist the whole power of the engine, applied alternately as compression and as extension.

The duty of the driving wheels is to transfer the power of the engine to the rails, that of the leading or truck wheels to guide the engine. The weight upon the drivers must be enough to secure the needed adhesion, and that upon the truck enough to

control the movement of the machine upon curves. The greatest improvement that has been made recently is the "Bissell Truck," which, instead of turning upon its own centre, turns upon a centre between its own and that of the forward driving wheels. By this arrangement, when the engine is upon a curve all the axles are practically radial, while upon the straight lines they are kept in much better parallelism than in the case of the ordinary truck. This allows of traversing sharp curves with great ease and with perfect safety, and reduces very much the wear upon the flanges of the wheels, and consequently upon the rails. By means of Mr. Bissell's plan a single pair of truck wheels is applied to heavy freight engines, with a long wheel base, allowing them to run with great ease even upon curves of small radius. By the connection between the compensating levers of the driving wheels and the truck, the effect of irregularities in the track is so distributed as to be much less felt upon the engine than in the ordinary arrangement. The system of Mr. Bissell has also been applied to tenders with excellent results, the forward truck being replaced by a single pair of wheels, turning upon a centre half way between the forward axle and the swing beam of the back truck.

The weight upon each pair of driving wheels varies from 8 to 16 tons, and in some cases even more than the latter has been employed, but certainly not judiciously. Upon the standard passenger engine of the Pennsylvania Railroad, 39,000 pounds are placed upon the drivers; or 9750 pounds on each driving wheel, and 24,100 pounds upon the truck. The standard ten-wheeled freight engine upon the same road, has 48,000 pounds on the drivers, or 8,000 pounds upon each wheel, and 20,500 pounds upon the truck. The heavy eight-wheeled connected freight engines of the "Consolidation" pattern have 75,640 pounds upon 8 wheels, or 9455 pounds per wheel, and 10,560 pounds upon the truck. The Baltimore and Ohio "Camel" engine has 28 tons upon 8 wheels, or $3\frac{1}{2}$ tons upon each wheel. The weight upon a single pair of wheels is frequently less in the heavy freight engines than in those employed for passenger trains; and adding to

the latter the high speeds at which they run, it will be found that the freight engines are much less severe upon the track than those employed for passenger service.

ADAPTATION OF THE LOCOMOTIVE.

In certain cases the application of the power upon railroads is necessarily bad ; for example, when the trains are very much heavier in one direction than in the other, as we are obliged to use the same engine both ways, because when it arrives at one end of the road it must go back again. So, too, when the traffic requires to be worked chiefly in the direction of ascending grades, we employ a much heavier engine to ascend with the load than is needed to descend with empty trains. The best adaptation of locomotive power to any system of grades will be that which shall render the mileage of engines a minimum ; and this will be done as nearly as possible by employing locomotives, the strength of which shall be proportional to the resistance to be overcome.

The capacity of an engine depends, of course, upon its adhesion, supposing the furnace, boiler, and the means for the distribution of the steam to be ample for the work, as they commonly are. Suppose the traffic upon a road to demand the employment of 1200 tons upon driving wheels. This may be obtained by 60 engines, with 20 tons upon the drivers of each ; and if there are two pairs of driving wheels, this will be .5 tons on each wheel. With eight-wheeled engines, the same amount upon each wheel would give 40 tons, requiring 30 engines. Upon the common eight-wheeled engine we have 10 tons upon the truck. With the "Consolidation" pattern we have but 5 tons upon the truck. There would, therefore, be the same amount of driving-wheel weight, in both cases, rolling over the track, and 300 tons more truck weight with the eight-wheeled engines. At $4.00 a day for wages of engineer and fireman on each engine, by reducing the number of engines from 60 to 30, we should save $120 per diem, or $36,000 per annum (300 days), which represents a capital of

$600,000. If the common form of engine costs $15,000, and the "Consolidation" $20,000 each, we save here again $300,000 of capital, making with the former amount $900,000. Calling the repairs of the 30 heavy engines the same as that of the 60 lighter ones, and fuel and other material the same, we should thus release a capital of $900,000 by the employment of the heavier class of locomotives.

It is often stated that the effect of long coupled engines upon the track, and conversely the wear and tear of the engine itself, is greater than that of lighter engines with fewer wheels. The facts, however, do not bear out such an assertion. It is stated by Mr. Latrobe, that the repairs of both track and engines are less upon roads doing a heavy freight business, with eight-wheeled coupled engines and no truck, than with the locomotive with four drivers and a truck.* It is hard to compare the cost of hauling freight upon different roads where the physical and commercial elements vary so much ; but there is no reason to suppose that engines of the "Consolidation" type, for the amount of work performed, will make the repairs of track or machinery any more than with the ordinary eight-wheeled engine ; while, as shown above, in other points they are vastly superior. The subject of the adaptation of locomotive power has received much more careful study upon the roads of Pennsylvania, Maryland, and Virginia, crossing as they do the great Appalachian ridge, than elsewhere. The heavy coal traffic of the anthracite region, too, has developed a powerful class of locomotive engines.

Different forms of the Locomotive.

The several forms of the locomotive, as commonly employed, are represented below. Fig. 156 is a four-wheeled tank engine, which may weigh from 10 to 20 tons, or from 2½ to 5 tons per wheel,

* See Appendix. Report to President of North Missouri Railroad, by B. H. Latrobe, Esq. 1866.

and is used for making up trains, and for switching work in depot grounds. The fore and aft rocking upon so short a wheel base at

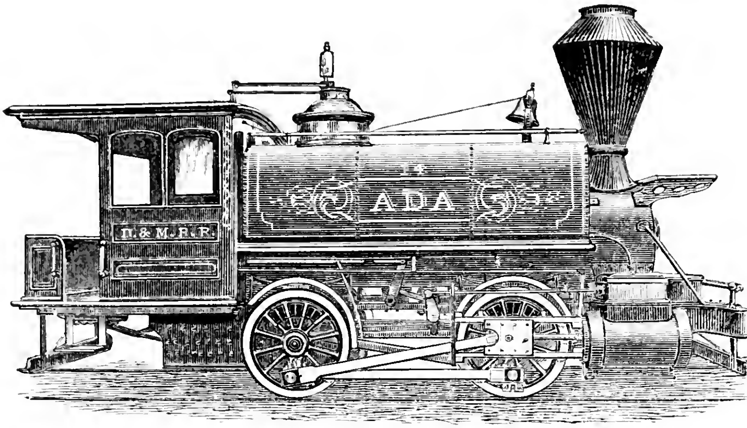


Fig. 156.

any but low speeds renders this form objectionable. This fault is remedied by the addition of the pony truck in front, shown in

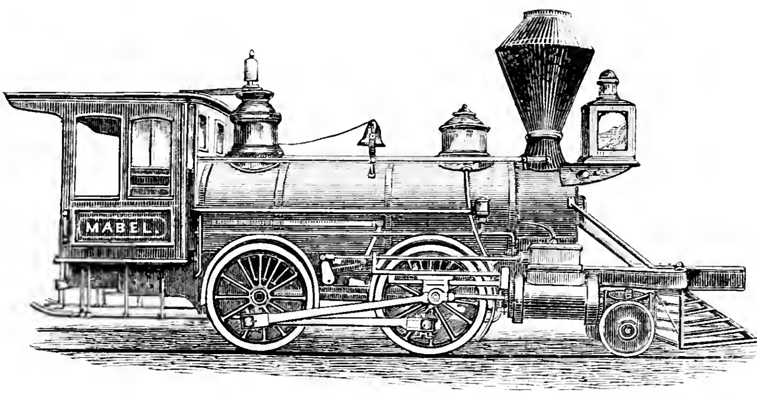


Fig. 157.

Fig. 157; and the addition of a similar truck behind gives the tank engine, designed by Mr. Hudson, of the Rogers Locomotive

Works, shown in Fig. 158, a most excellent machine for switching, construction, branch, and even light passenger service.*

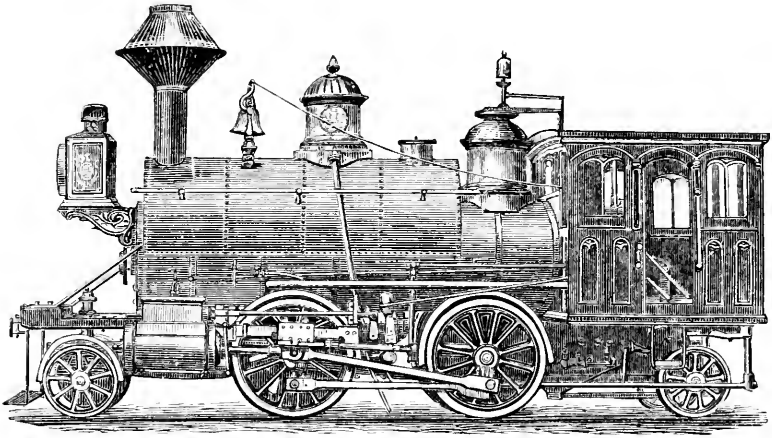


Fig. 158.

Fig. 159 shows a six-wheeled tank engine, which, upon a tolerably straight track, is a very effective machine, but upon sharp

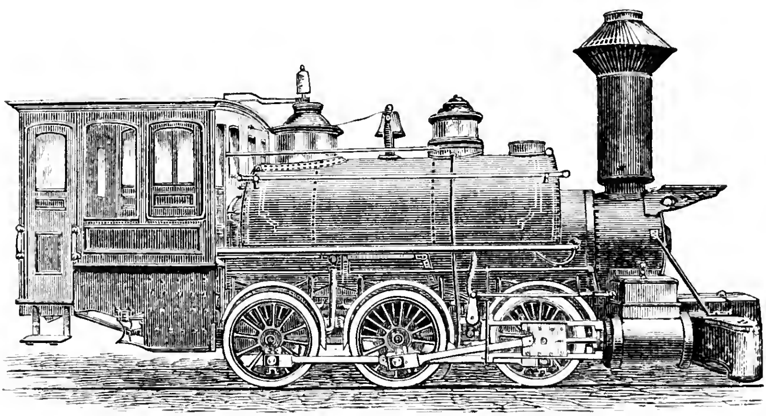


Fig. 159.

* The Portland Company build a four-wheeled engine for switching and for branch service, weighing 30 tons. Cylinders 15×24, wheels $61\frac{1}{2}''$, and $92''$ between centres. The front draw casting of the engine is pivoted under the

curves, or for considerable speeds, this form is very much improved by the addition of a single pair of truck wheels upon Mr. Bissell's plan, as shown in Fig. 161.

Fig. 160 shows the standard eight-wheeled passenger engine,

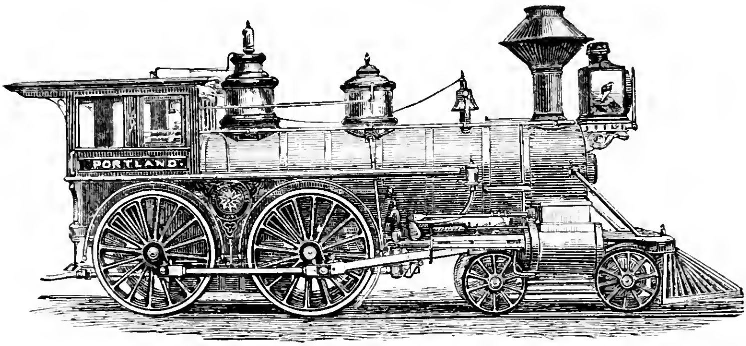


Fig. 160.

as built by the Portland Company, at Portland, Me. The cylinders are 16 × 22 inches, and the wheels 5 feet 7 inches in diameter, placed 84 inches from centre to centre. The truck wheels are placed 5 feet 8 inches from centre to centre. The distance from the centre of the truck to the centre of the first driving axle is 11 feet $5\frac{3}{4}$ inches. The total weight is 31 tons, with fire and water. The whole length of engine and tender is 50 feet. The tank holds 1900 gallons. A detailed specification of this engine will be found in the Appendix.

The eight-wheeled passenger engine, built by the Baldwin Locomotive Works of Philadelphia, has cylinders 16 × 24 inches. Driving wheels 66 inches, 8 feet centre to centre. Truck wheels 30 inches, placed 5 feet 8 inches centre to centre. Total

centre of the saddle, and not over the pilot, as in many of this class, thus enabling the engine to work more easily round curves, and reducing the wear upon the flanges on account of the pull being nearer to the centre of the engine. The back end is shortened, the engineer standing partly on the side of the boiler, thus reducing the over-hang, and so lessening the oscillation. This is not a tank engine, but it has an ordinary eight-wheeled tender.

wheel base 21 feet 9 inches. Length of engine over all 32 feet. Width 8 feet 4 inches. Height 14 feet 6 inches. Length of engine and tender over all 52 feet 4 inches. The weight of the engine, in working order, is 65,000 pounds, 42,000 of which are upon the drivers, and 23,000 pounds upon the truck. The tender, with 2,000 gallons of water and 3 tons of coal, weighs 42,000 pounds. Whole weight of engine and tender 107,000 pounds. The grate is 60 inches long and $34\frac{1}{2}$ inches wide. The fire box is 60 inches long, $34\frac{1}{2}$ inches wide, and 65 inches deep inside, having a surface of 97 square feet. The tubes are 144 in number, 2 inches in diameter, 11 feet 6 inches long, giving a surface of 866 square feet.

Fig. 161 is a pattern termed the "Mogul," recently introduced

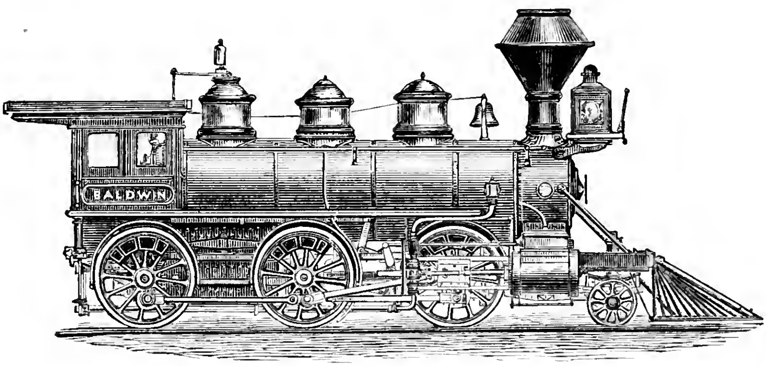


Fig. 161.

by Mr. A. Mitchell, division superintendent of machinery upon the Lehigh Valley Railroad, and intended for fast freight, or for heavy passenger trains. It is an exceedingly efficient machine, and is especially adapted to the passenger traffic for winter service in the Northern States, so large a part of the weight being available for adhesion. An engine of the above style upon the Lehigh Valley Railroad, with 61 inch drivers, cylinders 18×24 inches, grate $8\frac{1}{2}$ feet by 35 inches, the heating surface being 108 feet, tubes 178 of $1\frac{3}{4}$ inch, and 15 of 2 inch diameter, with a surface of

978 feet, or a total surface of 1087 feet, with 31 tons upon the drivers, has taken 75 coal cars, weighing 3 tons each, or 225 tons exclusive of engine and tender, over a 96 feet grade; and has hauled 11 passenger cars, with a load of 700 passengers, over 60 feet grades, at 14 miles an hour.

Plate XXX. represents side and end views of the "Mogul," as built by the Baldwin Locomotive Works, the dimensions being as follows: Cylinders, 17 inches in diameter and 24 in stroke. Driving wheels, 54 inches. Truck wheels, 30 inches. Fire-box, 60 inches long, 35 inches wide, and 67 inches high. Fire-box surface, 105 square feet. Grate, 60 inches long, 35 inches wide; area, 14.5 square feet. Tubes, 156 in number, 11′ $2\frac{3}{4}$″ long, and 2

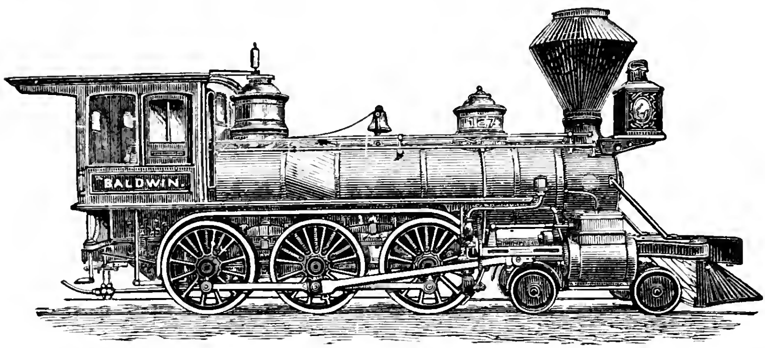


Fig. 162.

inches diameter. Tube surface, 917 square feet. The weight of this engine, in working order, is 70,000 pounds, of which 61,000 pounds are on the drivers and 9,000 on the truck. The tender, loaded, weighs 40,000 pounds. The total wheel base is 21′ 10″. The driving-wheel base is 14′ 6″, the front and back driving-wheels being flanged, the middle ones plain. The pony truck, having a swing bolster and radius bar, the rigid wheel base is the above 14′ 6″. Total wheel base of engine and tender, 43′ 2″. Total length over all, 53′ 8″. Length of engine, 36′ 7″. Height of engine over house, 10′ 7″; over stack, 14′ 6″. Width of buffer, 9 feet.

Fig. 162 shows the ten-wheeled freight engine, as built by the

Baldwin Locomotive Works, and shown in detail in Plate XXXI. The cylinders are 18″ × 24″. Driving wheels, 54 inches. Truck wheels, 26 inches. Driving-wheel base, 12 feet 8 inches. Total wheel base, 23 feet $4\frac{1}{2}$ inches. Whole wheel base of engine and tender, 44 feet 6 inches. Length over all of engine and tender, 54 feet 10 inches. Length of engine, 36 feet $2\frac{1}{4}$ inches. Height of engine over house, 10 feet 5 inches, and over stack, 14 feet 2 inches. Width, 8 feet 10 inches. The fire-box is 60 inches long, 35 inches wide, and 62 inches high, giving a surface of 94 square feet. The grate is 60 inches long by 35 inches wide, with an area of $14\frac{1}{2}$ square feet. Tubes, 135 in number, 12 feet $9\frac{1}{4}$ inches long and $2\frac{1}{4}$ inches in diameter, giving a surface of 944 square feet. The weight of the engine, in working order, is 77,000 pounds, of which 58,000 pounds are on the drivers and 19,000 upon the truck. The tender, loaded, weighs 44,000 pounds. This is, all things regarded, one of the best machines for doing a large freight business, and is very generally adopted upon the steep grades of Pennsylvania Railways.

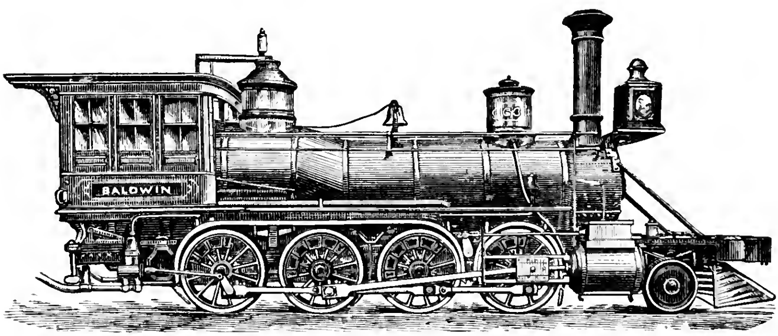


Fig. 163.

Fig. 163 represents the heavy eight-wheeled coupled freight engine, known as the "Consolidation" pattern, designed by Mr. Mitchell, of the Lehigh Valley Railroad, and built by the Baldwin Locomotive Works. It has four pairs of driving wheels, 49 inches in diameter, all coupled, and cylinders 20 × 24 inches. Grate, 120 inches long, and $34\frac{1}{2}$ inches wide. Fire-box, 120 inches long,

34½ inches wide, and 48 inches average height, giving a surface of 135 feet. Tubes, 173, of 2 inches diameter, and 12 feet 10¾ inches long, giving a surface of 1168 square feet. The weight of engine and tender, with feed, is 138,000 pounds. The engine alone weighs 90,000 pounds, 80,000 of which are upon the drivers and 10,000 upon the truck. The whole wheel base is 22 feet 4 inches, the driving-wheel base 15 feet, and the flanged or rigid driving-wheel base, there being no flanges on the front drivers, 10 feet 4 inches. The extreme length is 36 feet 3½ inches. Breadth, 9 feet 2 inches. Height, 14 feet 1 inch. Extreme length of engine and tender, 56 feet 11 inches.

An engine of this class has taken 33 coal cars, weighing, loaded, 8 tons each, or 264 tons, exclusive of engine and tender, over a 133 feet grade, with 125 pounds of steam, cut off at the third notch, or 16″, using sand to aid the adhesion. The common load is 28 cars. It has taken 81 empty coal wagons, weighing 3 tons each, or 243 tons, exclusive of engine and tender, over a grade of 145 feet, at a low speed, with 130 pounds of steam, cutting off at 16 inches, and using sand. It has taken 250 coal cars, empty, or 750 tons, over a 21 feet grade, and has started the same on the curve at Mauch Chunk, which is compounded from 6° to 13°, and thence down to 5°, the grade being 10 feet per mile. It has drawn 17 cars, 8 tons each, or 136 tons, exclusive of engine and tender, over a 133 feet grade, at about 8 miles an hour, with 120 pounds of steam, cut off at the fifth notch, using no sand, and in a snow-storm, on a bad rail. It has taken 115 loaded coal cars, or 920 tons, besides engine and tender, over 21 feet grades, at 16 miles an hour. It has taken 320 tons, besides engine and tender, up 8½ miles of 96 feet grade, and 4½ miles of 60 feet grade, in 58 minutes. With a dead pull at the bottom of a 77 feet grade, it took 504 tons up the grade and around a 9° curve, with 125 pounds of steam, cut off at 20 inches. Upon the Lehigh and Susquehanna Road, the superintendent of motive power states, that during the summer months, on a warm, dry rail, four of these engines will move 500 loaded cars daily from Nanticoke to Penobscot, or

up $8\frac{1}{2}$ miles of 96 feet grade, and $4\frac{1}{2}$ miles of 60 feet, on a line where there is hardly a tangent half a mile long, and where, in many places, a train of 40 cars will cover reversed curves, each engine thus making four round trips, or 104 miles, with 282 tons per engine. It is also stated by the same authority that these engines do not seem hard on the track, but curve freely, with but little wear, either upon the driving or the truck flanges, or upon the track. This engine has a furnace from 9 to 10 feet long, burns anthracite coal, and makes an abundance of steam.

Engines upon the Pennsylvania Railroad.

The standard passenger engine upon the Pennsylvania Railroad has four drivers, 66 inches in diameter. Cylinders, $17'' \times 24''$. Grate, $5'\ 5\frac{1}{2}''$ long, and $2'\ 10''$ wide, with an area of 15.4 square feet. The boiler shell is $49\frac{1}{4}''$ diameter, and $20'\ 2\frac{1}{2}''$ extreme length. The fire-box (steel), $6'\ 2''$ long outside, $3'\ 6''$ wide, $9\frac{1}{2}''$ high above centre, and $4'\ 6\frac{1}{2}''$ below; $7\frac{1}{2}''$ height of wagon top. Smoke arch, $50''$ diameter, out to out, and $33''$ long. Tubes, 142 (iron), $2\frac{1}{4}''$ diameter, and $11'\ 7''$ long. Surface, 959.5 feet. Total heating surface, 1058 feet. One dome, $30''$ outside diameter. Double exhaust nozzle, $3''$. Stack, $14\frac{1}{2}''$ diameter. Weight, (without feed) on drivers, 39,000 pounds, on truck, 24,100 pounds. Total, 63,100 pounds. One pump, 24×2 inches, and one No. 8 adjustable injector. The valves have $5''$ travel, face, $16\frac{1}{2}'' \times 8\frac{1}{2}''$. Steam ports, $15\frac{15}{16}'' \times 1\frac{1}{4}''$. Exhaust ports, $15\frac{15}{16}'' \times 2\frac{1}{2}''$. Outward lap, $\frac{3}{4}''$, inward lap, $\frac{1}{64}''$. Eccentric throw, $4\frac{1}{8}''$ or $4\frac{1}{5}''$. The engine has a full truck, with 30 inch wheels, placed $5'\ 8\frac{1}{4}''$ between centres. From centre pin to back axle is $19'\ 7\frac{1}{4}''$, and the total wheel base, $22'\ 5\frac{3}{8}''$.

The standard freight engine, upon the same road, has six drivers, $54\frac{5}{8}''$ diameter. Cylinders, $18'' \times 22''$. Grate, $61''$ long, $35''$ wide, with an area of 14.8 square feet. The boiler shell is $49\frac{3}{8}''$ diameter, and $21'\ 5\frac{1}{2}''$ long. The fire box is $69\frac{1}{2}''$ long outside, $42\frac{1}{8}''$ wide, $9''$ above centre, and $55\frac{1}{2}''$ below; height of wagon top, $8''$.

Smoke arch, 50″ outside diameter, and $33\frac{1}{4}$″ long. Tubes, 119 (iron), $2\frac{1}{2}$″ diameter, 12′ $9\frac{9}{16}$″ long, surface 996.1 feet. Total heating surface, 1096.1 feet. One dome, 30″ outside diameter. Double exhaust nozzle, $3\frac{1}{2}$″ × $2\frac{1}{4}$″. Stack, 18″ diameter. Weight, (without feed) on drivers, 48,000 pounds. On truck, 20,500 pounds. Total, 68,500 pounds. One pump, 22 × $2\frac{1}{4}$ inches, and one No. 8 automatic injector. The valves have 5″ travel, face, $16\frac{1}{2}$″ × $8\frac{1}{2}$″. Steam ports, 16″ × $1\frac{1}{4}$″. Exhaust port, 16″ × $2\frac{1}{2}$″. Outward lap, $\frac{3}{4}$ inch, inward lap, $\frac{1}{16}$. Eccentric throw, 5″, and diameter, 13″. Full truck, 5′ 8″ between centres. Centre pin to back axle, 20′ 10″. Whole wheel base, 23′ 8″

The mountain passenger helper in use upon the 96 feet grades is similar to the passenger engine above, except that the cylinders are 18″ × 24″, and wheels 5 feet, and the boiler a little larger. The mountain ten-wheeled freight engine has 4 feet drivers, and a larger boiler than that of the freight engine above. For local and fast freight trains a modification of the above passenger engine is used, differing only in the diameter of the drivers, which are five feet, and in the boiler, which is larger. For shifting, a six-wheeled engine is employed, with cylinders 15 × 18 inches, and 44 inch drivers.

The eight-wheeled tenders have tanks for 2400 gallons of water, and room for $3\frac{1}{2}$ tons of coal, weighing, empty, 19,750 pounds, with water, 39,900, and with fuel, 46,750 pounds.

The maximum load for a first-class passenger engine, from Altoona to Gallitzin, the average grade being about 90 feet per mile, is 5 cars ; and for a first-class freight engine, 11 cars. The freight train has generally 33 cars, and has two or three helping engines, two being placed in front and one behind. For passenger trains, one or two helping engines are put on in front. From Conemaugh to Gallitzin, the grade being 53 feet per mile, the passenger train for one engine consists of 7 cars, and for a freight engine, 17 cars.

Engines upon the Baltimore and Ohio Railroad.

The Baltimore and Ohio "Camel" weighs 28 tons, has four pairs of 42″ wheels coupled, with cast iron chilled tires. Cylinders, 19″ × 22″. Tubes, 101 of iron. It carries 100 pounds of steam, and cuts off at 75 per cent. It will haul 8 eight-wheeled cars loaded, or 150 tons in all, besides engine and tender, over the 116 feet grades, at 10 or 12 miles an hour. The grades over the Kingwood Tunnel were 520 feet per mile, and in some places 560 feet. The "Camel" hauled the iron for the western part of the road over these grades, the load being two flat cars, or 35 tons in all, exclusive of engine and tender, at 10 miles an hour.

The ten-wheeled engines, designed by Henry Tyson, for the same road, weigh, with water, 30 tons. They have six 50 inch drivers, all in front of the furnace, the back ones only being flanged. Cylinders, 18 × 24 inches. Truck wheels, 28 inches, placed 68″ centre to centre. Boiler, 48″ diameter at rear, and 46″ at front, $14\frac{1}{4}'$ long. Tubes, 125, $2\frac{1}{8}''$ and $2\frac{3}{8}''$ diameter, and $14'\ 3\frac{1}{2}''$ long. The load for these engines, upon the 116 feet grades, is 7 eight-wheeled cars, loaded.

Other Engines for Heavy Work.

The engines made by Baldwin, and used by Mr. Ellet upon the Mountain Top Track, referred to in Chapter III., the average grade for 12,500 feet being 257 feet, and the maximum 295 feet per mile, had outside cylinders $16\frac{1}{2} \times 20$ inches. Six coupled wheels 42″ inches, the whole wheel base being 9′ 4″. The weight, with 100 cubic feet of wood and 100 cubic feet of water, was 55,000 pounds. The load taken was two eight-wheeled passenger cars, and one eight-wheeled baggage car, or three eight-wheeled freight cars, weighing in all about 45 tons, exclusive of engine, and running at $7\frac{1}{2}$ miles an hour.

The engine, designed and built by Mr. Reuben Wells, for working the Madison grade, upon the Jeffersonville, Madison, and In-

dianapolis Railroad, which is straight, but rises 394 feet in $1\frac{1}{4}$ miles, or 320 feet per mile, has five pairs of coupled wheels, 3′ 8″ diameter, and no truck, three pairs being in front and two pairs back of the furnace. Cylinders, $20\frac{1}{8}'' \times 24''$. Grate area, $15\frac{3}{4}$ square feet. Fire surface, 116 square feet. External tube surface, $1262\frac{1}{4}$ square feet. Total surface, $1378\frac{1}{4}$ square feet. Distance between centres of first and second pairs of wheels (from the front), 4′ 6″; second and third, 4′; third and fourth, 7′ 6″; fourth and fifth, 5′. Whole wheel base, 21′. Steam ports, $1' 6'' \times 1\frac{1}{8}''$. Exhaust, $1' 6'' \times 2\frac{1}{2}''$. Valves, $\frac{1}{2}''$ outside, and $\frac{1}{16}''$ inside lap; $\frac{1}{8}''$ lead in full gear, cutting off at $22\frac{1}{8}''$. Two exhaust nozzles, $3\frac{5}{8}''$ diameter. Axle journals, 8″ long; first and second sets, $6\frac{1}{4}''$ diameter; third, $7\frac{3}{4}$; fourth and fifth, $5\frac{3}{4}$. Tires, all 6″ wide, those of the leading and trailing wheels being set $\frac{1}{2}''$ nearer than the regular gauge, to allow side play, all the wheels being flanged. Besides this, the axle of the fourth pair has $\frac{1}{2}''$ lateral play in the boxes, and that of the fifth pair, a play of $1\frac{1}{4}''$. The back ends of the trailing springs are connected by a cross lever. Two pumps, $3\frac{7}{8} \times 10$, and one No. 8-injector. Total weight of engine, with tanks full, and $1\frac{1}{4}$ cords of wood, is 112,000 pounds. Average weight, 108,000 pounds.

This engine has drawn six loaded cars, weighing in all 229,200 pounds, exclusive of engine and tender, up the grade (320 feet per mile) at 5 miles an hour, with a boiler pressure of 145 pounds, cutting off at from 21 to $22\frac{1}{8}''$. The usual load is 200,000 pounds, exclusive of engine and tender, at 6 miles an hour with 130 pounds of steam in the boiler, cutting off at 20″. Upon one occasion a train weighing 196,800 pounds was being taken up, the valves cutting off at $22\frac{1}{8}''$, when the steam was allowed to fall gradually to 113 pounds, when the engine stopped, the speed being 3 miles an hour. The regulator being left open, the blower was turned on until the steam reached 125 pounds, when the engine started and took the load up the remainder of the way.

The tractive power for each pound of effective steam pressure is —

$$\frac{(20\frac{1}{2})^2 \times 24}{44}, \text{ or } 220.92 \text{ pounds,}$$

The available adhesion upon the 320 feet grade, at 108,000 pounds on the drivers, is 107 759 pounds, or 241 pounds less than the weight, since the force pressing at right angles to the plane is to the whole weight as the base of the inclined plane representing the road is to its actual length, or to the hypothenuse.

The resistance of the 229,200 pounds upon the 320 feet grade would be —

Gravity, . .	$\frac{229.200 + 108.000}{16.5} =$	20,436
Engine, . .	48.21 tons, at 18 pounds, . . .	868
Train, . .	150.50 tons, at 10 pounds, . . .	1,505
	Total, ...	22,809 pounds.

This, divided by 220.9, gives $103\frac{1}{4}$ pounds per square inch as the effective pressure required on the pistons. The adhesion would be $22\frac{3}{4}$ per cent. of the weight upon the wheels.*

Counter-pressure Steam.

Since 1865 the use of counter-pressure steam in the locomotive cylinder for controlling the motion of trains running down

* The reader is referred to Zerah Colburn's work, "Locomotive Engineering and the Mechanism of Railways," published in Glasgow, Edinburgh, and London (which, though never completed by the author, is a most valuable work), for illustrations of numerous locomotives adapted to exceptional grades and curves. In this country, the Double Engine "Janus," built on Mr. Fairlie's plan, somewhat modified by William Mason, of Taunton, has attracted much attention. This locomotive is, in fact, two six-wheeled engines, placed back to back, each set of six wheels turning on their own centre. The whole weight is, of course, available for adhesion, and the engine is able to run easily round sharp curves. This plan saves one engineer and fireman, and has no useless weight to transport. Whether so cumbrous a piece of mechanism will be serviceable, except in cases where steep grades, sharp curves, and constant and heavy loads are combined, is doubtful.

long and steep inclines, has been thoroughly examined and widely adopted in Europe. From the valuable memoir of M. Le Chatelier, a clear idea of this operation may be had.* A locomotive cannot run forward with the valve gear reversed for more than a few minutes without heating the cylinders, and cutting the rubbing surfaces, from lack of lubrication. In the reversed working, the cylinders act as pumps, drawing in the heated gases from the smoke arch, and forcing them into the boiler. The plan of M. Le Chatelier consists in introducing a small jet of hot water from the boiler into the base of the blast pipe. This jet being discharged at boiler pressure into the atmospheric pressure of the exhaust passages becomes steam at atmospheric pressure, and thus a moist vapor is drawn into the cylinders behind the piston, instead of the heated gases from the smoke arch. The rubbing surfaces are thus lubricated, and heating and cutting is prevented. The last part of each stroke of an engine running thus is made against the full boiler pressure, which resists the movement of the piston, and thus acts as a powerful force to resist the movement of the wheels, making the weight of the engine available as brake power. By this means the engineer can at once employ all the load upon the drivers as a brake, with ease and with perfect safety. The only mechanism necessary is a tube about an inch in diameter, and a cock or valve.

The Injector.

Besides the ordinary pump for supplying the locomotive boiler with water, many engines are now fitted with the Giffard Injector. By means of this appliance a jet of steam is drawn from the boiler, and being condensed by a stream of water, with which

* Use of Counter-pressure Steam in the Locomotive Engine, as a Brake, by M. L. Le Chatelier, Ingénieur en Chef des Mines. Translated from the author's manuscript, by Lewis D. B. Gordon, F. R. S. E., 1869. The reader is especially recommended to peruse this memoir, as being a very concise exposition of a subject of importance.

it comes in contact, it gives to the water a velocity sufficient to throw it into the boiler, against the very steam pressure that produced the movement.* The supply of water may, by the injector, be maintained while the engine is at rest, — an advantage which will be felt during the winter months, when engines upon the northern railways are so frequently blocked by snow, and unable to work the pumps. A common practice is to fit locomotives with one pump and one injector, the size of the latter in ordinary use being that known as No. 8.

* The explanation of this apparent anomaly, as well as a complete description of the instrument as applied to the locomotive, may be seen in the Journal of the Franklin Institute, for 1868.

CHAPTER XVIII.

THE ROLLING STOCK.

THE rolling stock employed in this country consists of the standard passenger car, the palace, or drawing-room car, and the sleeping car, the last two being merely modifications of the first, affording more room and extra conveniences, and weighing more. These cars consist of a long body, well framed, and trussed vertically, the whole being placed upon two separate trucks, at the ends, each turning on a centre of its own, by which the passage of curves is made easy. Each truck has four, six, or even eight wheels, the latter being, in fact, two short trucks connected by a frame, upon the centre of which one end of the body of the car rests.

The bearing of the body of the car upon the truck is by means of springs placed upon a cross beam, the latter being suspended by links from the truck frame, half or three fourths of an inch of lateral movement being allowed. Springs are also placed between the truck frame and the axles, so that the shocks are almost completely destroyed before reaching the body of the car.

DIMENSIONS AND WEIGHT OF CARS.

For the standard gauge of 4′ 8½″, the ordinary passenger car for 60 persons is 48 feet long, or including platforms, 54 feet; the width, 9′ 6″; height at sides, 7′ 10″, and at centre, 10′ 3″. The weight, empty, is 39,000 pounds, or 650 pounds per passenger; loaded, it weighs 46,980 pounds, or on each of 8 wheels 5,872 pounds, or each of 12 wheels 3,915 pounds.

A sleeping car, for 64 passengers, has a body 61 feet long, 8′ 10″ wide inside, 7′ 10″ high at sides, and 9′ 7″ at centre, has 12 wheels, and weighs, empty, 26 tons, or 812½ pounds per passenger; or loaded, 945½ pounds per passenger. The load on each wheel is 5,042 pounds. The sixteen-wheel western palace cars weigh, loaded, 78,500 pounds, or 4,907 pounds per wheel.

The mail, express, and baggage car is 45 feet long, 9′ 4″ wide, 7′ 4″ high at the sides, and 9′ at centre. It weighs 27,000 pounds, and will carry 12 tons; making a total weight, loaded, of 51,000 pounds, or 6,375 pounds on each of 8 wheels.

The freight cars in most general use have 8 wheels, upon which is placed a strong platform 27 feet long and 8½ feet wide, thus making the platform, or "flat" car employed for the transport of lumber, masts, carriages, and bulky freight generally. The "box," "house," or covered car differs from the above only in having a simple rectangular house 7¼ feet high at the sides, and 8 feet at centre, built upon the platform. This is used for the protection of such freight as will not bear exposure, as grain, hay, furniture, dry goods, hardware, small machinery, etc. The weight of such a car is 17,800 pounds, or loaded, 37,800 pounds. The weight on each wheel, if loaded, is 4,725 pounds.

For the movement of coal, cars are used either with 8 or with 4 wheels. The eight-wheeled car has a body 20 feet long, or 21′ 10″ over all, 7 feet wide, or 7′ 8″ over all, and sides 4 feet high; weight, empty, 13,440 pounds, loaded, 35,840 pounds, or 4,480 pounds per wheel. The four-wheeled car has a body 11 feet long, or 13 feet over all, and 6′ 7″ wide, or 7′ 5″ over all, and 4′ 3″ high. Weight, empty, 6,720 pounds; loaded, 20,160 pounds, or on each wheel 5,040 pounds.

The height, above the track, of the floor of the cars in common use is 4 feet, and the height of the centre of the draw-bar from 2′ 6″ to 2′ 9″; the latter being that recommended by the National Car Builders' Association for both passenger and freight cars. The whole height of a box car would thus be 12 feet above the rail, to which is to be added 3 feet more for the brake-

wheel, making 15 feet in all. The ordinary engine requires a height of 14½ feet above the rail. These dimensions are to be regarded in determining the height of overhead bridges and the doors of station-houses.

CAR WHEELS.

The wheels universally employed for cars in the United States are of cast iron, the most common diameter being 33 inches, and the weight varying from 480 pounds for light freight cars, to from 500 to 530 pounds for passenger cars, and from 580 to 600 pounds for extra heavy wheels for engine trucks. The tread is chilled to a depth of about half an inch, and the wheels are cooled slowly to insure a tough and strong body. The best wheels are made entirely from the strongest irons, without any mixture of old wheels. The life of such wheels is stated by Mr. Colburn to be not less than 150,000 miles; while they have run in Canada 160,000 miles, being still in good order.* The axle journal runs in a composition metal bearing, placed outside of the wheel, enclosed in a close fitting iron box, with a movable cover at the outer end. The bottom of the box is filled with waste, soaked with oil. By lifting the box with a jack-screw, the packing may be replaced and the brasses renewed.

CONING OF WHEELS.

As remarked in a previous chapter, the outer wheel upon a curve has a longer distance to travel than the inner one; and that the wheels may run without dragging or slipping, the tread is made slightly conical, so that the tendency of the truck to press against the outer rail brings a larger diameter upon the outer and a smaller one upon the inner rail. The amount of such coning must depend upon the radius of the curve and upon the gauge.

* American Locomotives and Rolling Stock. From Proceedings Inst. Civ. Eng. London, 1869.

The gauge may be regarded as constant, i. e., 4′ 8½″, and for curvature the average radius must be taken, since the cone cannot be exactly adapted to different radii. The cone of the engine tires made by the Portland Company, the flange of which is shown in Fig. 10, Plate XXVIII., is $\frac{1}{16}$″ in a width of nearly 4″ ($3\frac{15}{16}$″). The car wheels by the same company cone nearly double that amount. This is much less than the theory gives. Thus the rule given by Mr. Henck (Field Book, p. 90) for a radius of 600 feet, gauge 4.7 feet, and wheels 2.8 feet diameter, gives a cone of .011 feet per inch, or nearly half an inch in a 3½ inch tire, or about four times the quantity ordinarily found in practice. Of course the sharper the curve the more cone is required. The cone cannot be varied, but the lateral play of the wheels may be made more or less by altering the gauge of the rails. It is the custom upon many roads to widen the gauge upon curves of less than 6°, or 955 feet radius, from 4′ 8½″ to 4′ 9″.*

Axles.

No part of the rolling stock demands a more careful study than the design and making of the axles of cars. More disasters have occurred from broken axles than from any other defect in the rolling stock. The metal must not only be of the best quality, whether of iron or steel, but it must have the right form. The point where fracture almost invariably occurs is just inside the wheel; the fracture commencing by the formation of a minute crack at the angle where the journal joins the body of the axle. This crack works inwards until the axle is unable to bear the

* The adoption of loose wheels would of course remove the need of coning, and would thus save twisting the axles, and a great deal of oscillation upon straight lines. This, however, has never yet been done in a satisfactory manner. "In a perfect system," says D. K. Clark, "the wheels must be loose on the axles." "If the wheels were free," says Mr. Wm. Bridges Adams, "instead of being held fast laterally to the frame, they would find out for themselves the path of least friction, the torsion of the axles would cease, and oscillation of the bodies would cease also."

shocks to which it is subjected. The commencement of this crack is due to a lack of continuity in the fibres of the surface at the angle of the shoulder, by which the elastic play of the fibres is checked at that point. Rounding the angle has been found to add very much to the strength ; but when the shoulder is rounded by turning in the lathe, we cut the surface fibres and destroy their continuity. Hammering the axle into the proper form has produced much better results ; journals so made requiring from five to eight times as many blows with a hammer to break them as when turned with a square shoulder. Preservation of the continuity of the fibres is the end to be attained.* Mr. Joseph Anthony has examined this question, and in a valuable essay has illustrated the various forms which have been tried. The rail, he observes, is, by means of the wheel, the fulcrum, the outer end of the axle the short arm, and the inner half of the axle the long arm of a lever. The weight carried by the journal multiplied by the short arm, and the product divided by the long arm, or half axle, determines the power needed at the middle, which power is afforded by the stiffness of the axle. As the shocks received by the axle should be absorbed as equally as possible throughout the whole length, the diameter should decrease towards the centre. Mr. Anthony observes, that axles $3\frac{1}{4}''$ at centre and $4''$ at the bosses have borne the heaviest and fastest freight for 18 years without any appearance of damage, while many axles forged uniform throughout, have broken in half that time. All necessary changes in the diameter should be made gradually, so that the vibrations in the metal may not be checked suddenly, as they always are at square shoulders. "A square neck or shoulder," says D. K. Clark, "is an incipient fracture ; it is the beginning of a break." "In all cases," says Mr. Anthony, "where vibrations are concentrated at an angle, and granulation and fracture in consequence take place, the metal in all of the other parts remains in its original and perfect condition. By not being

* See Rankine, in Trans. Inst. Eng. Scotland, 1862–3, and Bramwell, in Report Brit. Ass'n. 1869. Exeter Meeting.

in the unfavorable position, it has not been subjected to the injurious action."

"To prescribe a remedy for this defect in the form of axles, it is evident that the wheel, and the axle on which it is placed, should each or both be so formed that when connected together, the same conditions as respects angles and abrupt changes in form and strength, shall apply to the structure as a whole, as to axles themselves. The trouble is, that while the wheel and axle have been acting as one and the same structure, they have not been formed as though they thus acted, but instead, each has been so made as to combine in itself the greatest strength without reference to their close union and joint action."

The form of axle embodying these requirements, as given by Mr. Anthony at the close of his essay, is that shown in Fig. 164,

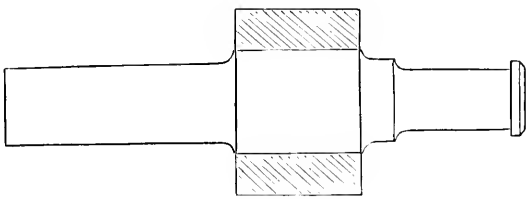


Fig. 164.

in which form the angles, by which the continuity of the vibration is broken and fracture thus invited, are avoided. The shocks in such an axle are spread over the whole length, instead of being concentrated at special places. If this form be made altogether by hammering instead of by cutting, so much the better.

With regard to the granulation or crystallization of the metal of railway axles, we find the same difference of opinion that always presents itself upon this much discussed question, in which not only the opinions of engineers vary, but the statement of facts, even to flat contradiction.

The standard dimensions for passenger car axles on the Pennsylvania Railroad, are as follows: Length, $6' \ 10\frac{3}{4}''$; diameter at centre, $3\frac{15}{16}''$; diameter next to wheel fit, $4\frac{3}{8}''$; wheel fit, $4\frac{1}{4}''$; journal,

$3\frac{1}{4}''$, and $7''$ long; collar at end, $\frac{1}{2}''$. For freight axles, length, $6'\ 10\frac{3}{4}''$; diameter at centre, $4\frac{1}{8}$; next to wheel fit, $4\frac{1}{8}$; wheel fit, $4''$; journal $3\frac{1}{4}''$, and $7''$ long; collar $\frac{1}{2}''$ long, and $3\frac{7}{8}$ diameter. Two steel axles out of every hundred are tested under a drop, weighing 1,640 pounds, the axle being supported upon bearings 3 feet apart. Passenger axles are required to stand five blows at 30 feet, and freight axles five blows at 25 feet, the axles being turned after each blow. The experiments of Mr. Stummer, of the Northern Railway of Austria, show Bessemer steel axles to bear 1,000,000 foot pounds of blows applied to the alteration of molecular arrangement, while iron axles broke with from 350,000 to 500,000 foot pounds. Bessemer steel axles were found to show double the cohesive strength of the best Styrian or English iron axles; and to obtain the same deflection as with iron, the number of foot pounds exerted by the falling weight had to be doubled.

Car Couplings.

Railway managers are divided as to the merits of a close and firm coupling between the different cars of a train. In Europe the screw and buffer, by which the wagons are made, as it were, into one long, but flexible car, is generally adopted. In this country the loose shackling link has been very commonly used. By the latter, the oscillation of the car body is entirely unchecked, and a disagreeable shock is given to the passengers when the slack between the cars is taken up by the engine starting too suddenly. The cases in which a weak coupling has broken, thus preventing one part of a train from dragging the rest to destruction, are quite as numerous as those in which the strength of the coupling has been the means of holding a car from ruin. The improved trussed platform and automatic coupling of E. Miller, now in use upon many roads, is certainly a great improvement upon the old system. By the old method the platforms are *below* the sills of the cars, and the buffer below this again. This arrangement breaks the line of the sills, so that the slightest collision

will break the platforms, and almost always cause one car to strike the opposite one *above* its sills, thus producing what has been termed "telescoping," by which the floor of one car is forced through the next car just above the floor, cutting off the seats, and killing and mutilating the passengers. It is now generally admitted that the platforms and the buffers should be *in*, and not *below*, the line of the sills. In the method of Mr. Miller this is done. The coupling hook is attached to the draw spring, the same as the ordinary draw head, and at the same height above the track, but in such a manner that the outer end is free to move laterally and vertically a sufficient distance. The coupling hook projects beyond the platform, and a stop prevents accidental uncoupling. When two cars are brought together the coupling hooks, from their shape, push each other aside until the buffers are compressed hard on the buffer springs, and the points of the hooks having passed each other, the hooks are carried forward by their main-springs, and the coupling and compression both affected automatically at the same time, and without the use of links or pins. When two cars are thus coupled, the head of the hook of each car is under the buffer beam of the next car, and the platforms are only about four inches apart. Thus one platform cannot be forced over the other, while the compression makes the train run steadily, and prevents all jerking in starting and stopping. By this arrangement the platforms are held in a plane with the sills of the cars, oscillation is prevented, and telescoping is impossible. This coupling will not separate accidentally, but may be uncoupled by the levers from the platform without danger to the brakeman, and without shutting off steam to make a "flying switch." When the motion of a loose coupled train is suddenly checked, either by a derailment, or by sudden breaking up, the rear cars tend to crowd up against those in front, and if the speed is great, telescoping results. This action is almost, if not entirely, prevented by the close coupling of buffers which lie in the line of the sills, as above described.

BRAKES.

The ordinary method of stopping a train is by brakes, applied by hand to each set of wheels. This is done by the brakeman, who stands between the cars, first to the front of one car and then to the rear of the adjacent one. The retarding force is thus applied irregularly, and without any system, to the discomfort of passengers, and often to the damage of the cars. A correct system should apply a sufficient power, gradually and uniformly, to the whole train. Many plans for retarding trains have been contrived, some of them self-acting, others applied at the engine. The Creamer Brake, by which the blocks are forced upon the wheels by strong springs, and when not wanted, are forcibly drawn away and locked, is a simple and efficient contrivance. The Loughbridge Brake is worked from the back driving wheel of the engine, the flange of which runs into the groove of a pulley, which has connection, by means of a chain, with the brakes under the cars. The ordinary hand brakes are connected with the same apparatus, the two systems, however, being distinct in operation.

The Westinghouse Atmospheric Brake consists of an upright direct-acting air pump, placed on the right hand side of the engine, between the driving wheels, taking its steam from the boiler, and pumping air into a cylindrical reservoir placed beneath the foot-board. The pump is entirely self-acting, and is, in fact, a medium between the pressure of steam in the boiler and the pressure of air in the reservoir. When the boiler pressure rises, the pump works more actively, until the pressure of the air in the reservoir rises to correspond ; or, if air is drawn from the reservoir to apply the brake, the pump at once begins to work more vigorously, without attention from the engineer, to make up the deficiency of pressure in the reservoir. Under each car is placed a small cylinder, the piston of which acts directly on the lever commonly used for the hand brake, but in no way interfering with braking by hand. The condensed air is carried to these

cylinders by a $\frac{3}{4}$ inch gas pipe, running beneath the cars the entire length of the train, a connection being made at the shackling of the cars by rubber hose, so coupled that when the parts are united the air passes freely along; but should the train be broken at a coupling, a valve closes, and prevents the escape of air, the brakes remaining effective. By this arrangement the entire management of the train is placed in the hands of the engineer, who can at once apply every brake in the train with the utmost force, or as quickly release the same. Trains have been stopped while running at speeds from 30 to 40 miles an hour, by the above brake, in as short a distance as 400 feet. A train of six cars, running down a 96 feet grade, upon the Pennsylvania Railroad, at 30 miles an hour, was stopped in a length of 420 feet. Safety, comfort, and economy are at once attained by applying the retarding power to all parts of the train alike; and this is done by the Westinghouse Brake.

Retarding by counter-pressure steam has been noticed in a former chapter.

CHAPTER XIX.

RAILWAY STATIONS AND SHOPS.

The buildings required for operating a railway consist of terminal passenger and freight stations, way passenger and freight stations, engine houses, wood or coal sheds, water tanks, and repair shops for engines and cars; the latter having sufficient capacity, upon large roads, for the manufacture as well as maintenance of rolling stock. The terminal passenger-house will be located at the most convenient point of access to the persons using it, regard being had to connection with the other routes of travel, hotels, etc. The terminal freight buildings, requiring a large amount of room for distributing and making up the trains, will be more removed from the central parts of cities. The engine and car houses and repair shops may be placed where land is less expensive than within city limits, and so distant from dwelling-houses as not to cause inconvenience by smoke and noise. Wood-sheds, tanks, and turn-tables will be at the engine-houses; scales for the weighing of cars, at the freight buildings. The departments for arrival and departure for the passenger service will be most properly accommodated in the same buildings; but for freight, they may often be separate.

The most common form for an engine-house is circular, or rather polygonal, the turn-table being placed in the centre. The diameter of such a building will, of course, depend upon the number of engines to be accommodated. The circumference of a 60 feet table is 188.5 feet, which would allow of 15 divisions of $12\frac{1}{2}$ feet each, very nearly. Thus, if the front of the building came

directly up to the edge of the table, and if a depth of 75 feet was required from the front to the back, a diameter of 210 feet would be needed for 15 engines. A yard, however, 40 or 50 feet deep, between the table and the front of the building, is very convenient, and is commonly employed. With a table 60 feet in diameter, and a yard 20 feet deep, the doors being 12 feet wide from centre to centre of post, we have accommodations for 25 engines, and the building being 75 feet deep, the whole diameter is 250 feet; with a 30 feet yard, 30 engines, and diameter 270 feet; with a 40 feet yard, 35 engines, and diameter 290 feet; and with a 50 feet yard, 40 engines, and a diameter of 310 feet.

At way stations, the freight and passenger-houses and the wood and water station may be combined; the plan and size depending upon the location and importance of the station. The relative position of the tank, wood-shed, and passenger-house should be such, that when the tender is at the proper place for receiving its supplies, the centre of a passenger train, of ordinary length, shall be at the passenger door.

The number of engines leaving the end of a road determines the amount of water necessary at the principal stations; and the character of the road and of the traffic fixes the location and size of the way water-stations. The amount of traffic being pretty equally distributed over the length of the road, the tanks should be placed at equal equated distances. Thus the engines will need to water at closer points upon steep grades than upon level roads. The tanks may be filled by hand power, by steam, by water, or by wind. Oftentimes high springs will furnish the necessary supply, without the application of artificial power.

The valve employed for drawing the water off from the tanks to the engines should be easily lowered and raised, and should admit of being moved around laterally, to reach the hole in the tender at points varying from four to six feet, thus avoiding the moving of the train or engine back and forth. It should also have a water-tight joint in any position, preventing waste of water and accumulation of ice about the tank.

The locomotive tank holds from 1500 to 2500 gallons, and the engine uses from 25 to 50 gallons per mile for light trains, and from 75 to 100 gallons for heavy trains. The amount will, of course, be greater for working steep grades than for roads level, or nearly so. (For the size, capacity, and construction of tanks for water stations the reader is referred to Mr. Trautwine's Civil Engineer's Pocket Book.)

A very complete description will be found in the Franklin Journal, for 1871, of the West Philadelphia shops of the Pennsylvania Railroad, by the engineer, Mr. Joseph M. Wilson. The large and complete works at Altoona, upon the same road, are especially worthy of careful examination by the engineer.

Turn-tables are made either of wood or iron, trussed above or below the level of the rails. The best form in use at present consists of a pair of cast or wrought iron girders, one under each rail of the track, the centre bearing being an adjustable steel pivot, or a set of conical steel rollers, or spherical balls. The ends rest upon wheels running upon a circular track, though these bear but very little of the load, being chiefly employed to steady the table as it turns.

The Keystone Bridge Company, and the Detroit Bridge and Iron Works, both make a table of boiler plate. For a 50 feet table the girder has a top flange $8 \times \frac{1}{2}$ inches throughout, to which is added for the middle third of the length a second plate of the same size. Such a table is $3\frac{1}{2}$ feet deep at the middle, the lower flange running up to the ends, to give the whole a fish-bellied form. The web is of $\frac{1}{4}''$ iron, stiffened at intervals of 5 feet by vertical ribs, which also form the splice for the joints. The angle irons, connecting the flanges with the web, are $3'' \times 3'' \times \frac{3}{8}''$. The two girders are connected by a stiff cross-bracing of angle iron, and at the centre by wide $\frac{1}{2}$ inch plates, forming the connection with the bearing pivot. Such a table, with the gearing, weighs from 10 to 12 tons. They are strong, stiff, durable, and easily moved. The table made by William Sellers and Company, of Philadelphia, is the same in form as those above described, but the girders are of cast iron.

CHAPTER XX.

RAILWAY MANAGEMENT.

"All that is required to render the efforts of Railroad Companies in every respect equal to that of individuals, is a rigid system of personal accountability through every grade of service." — D. C. McCallum, 1856.

Organization and Duties of Employees.

Railway Management may be divided generally into the two departments of Finance and Operation; the first embracing the Capital Account, with the care of its Stock, Bonds, Interest, and Dividends, and the second the transportation of passengers and freight, and the maintaining of the road and its equipment in a state of the utmost efficiency. The General Officers are a President, one or more Vice-Presidents, a Treasurer, Secretary, and an Attorney. The operating department is both commercial and mechanical, and its organization is made accordingly. This varies with the extent of the road, and the amount of business to be transacted. In some cases a General Superintendent, appointed by and accountable to the Board of Directors, has entire control of the operating department, having assistant or division superintendents upon different parts of the road, while subordinate to these latter are the Roadmasters, Master Mechanics, etc. In other cases the authority is divided primarily between a Master of Transportation, an Engineer in charge of the road, bridges, and buildings, a Superintendent of Motive Power and Rolling Stock, an Agent for the purchase of all supplies required for working and repairing the road, and a Superintendent of the Telegraph; each of these

officers being accountable to the Board of Directors. The office of Master of Transportation is commonly divided between a General Freight Agent and a General Passenger Agent.

Upon a road 50 miles long, running but few trains, a superintendent may not only direct the business generally, but also in detail, and may be acquainted with all of the employees. Upon a road 500 miles long a very different state of things exists. In the latter a thoroughly perfected system is indispensable. In an elaborate report, made in 1856, by D. C. McCallum, to the stockholders of the New York and Erie Railroad (now the Erie Railway), the following general principles are laid down as governing the formation of an efficient system of operation: *First.* A proper division of responsibilities. *Second.* Sufficient power conferred to enable the same to be fully carried out, that such responsibilities may be real in their character. *Third.* The means of knowing whether such responsibilities are faithfully executed. *Fourth.* Great promptness in the report of all derelictions of duty, that evils may be at once corrected. *Fifth.* Such information to be obtained through a system of daily reports and checks that will not embarrass principal officers, nor lessen their influence with their subordinates. *Sixth.* The adoption of a system, as a whole, which will not only enable the general superintendent to detect errors immediately, but will also point out the delinquent.

"A system of operation," says Mr. McCallum, "to be efficient and successful, should be such as to give to the principal and responsible head of the running department a complete daily history of details, in all their minutiæ. Without such supervision the procurement of a satisfactory annual statement must be regarded as extremely problematical. The fact that dividends are earned without such control does not disprove the position, as in many cases the extraordinarily remunerative nature of an enterprise may insure satisfactory returns under the most loose and inefficient management."

In the report above referred to, the following organization

is laid down for the officers acting directly under the General Superintendent : —

1. Division and Branch Superintendents.
2. Masters of Engine and Car Repairs.
3. Car Inspectors.
4. General Freight Agent.
5. General Ticket Agent.
6. General Fuel Agent.
7. Superintendent of Telegraph.
8. Foreman of Bridge Repairs.

The Division Superintendents are held responsible for the successful working of their several divisions, and for the maintenance of proper discipline and conduct of all persons employed thereon, except such as are in the employment of other officers acting under directions from the general office. They possess all the powers delegated by the organization to the General Superintendent, except in matters pertaining to the duties of the General Ticket and Freight Agents, the Fuel Agent, Superintendent of Telegraph, and Masters of Engine and Car Repairs. They have authority to change, by telegraph or otherwise, the movement of trains upon their divisions from the times specified in the tables.

Masters of engine repairs are held responsible for the good condition of the engines and machinery in shops, and the cost of their repairs. It is their duty to make frequent and thorough inspection of the engines, so as to guard them from accidents and injuries which may result from the want of seasonable and trifling renewals, and also to see that the engines are otherwise in efficient condition for use. They are also required to report to the Division Superintendents all cases they may discover of abuse or maltreatment of locomotives by engineers or despatchers.

For the more thorough supervision of cars, while in transit, the road is separated into divisions, according to its length, and an inspector appointed for each division, whose duty it is to examine both passenger and freight cars, to see that they are kept in

good condition, having special regard to the reduction of friction, making all small repairs, such as may be performed without aid from the force employed in shops. The moment any car is judged unsafe it is condemned, and the attention of the Master of Car Repairs is called to the fact.

The duties of the General Freight Agent are, with the approval of the President or General Superintendent, to make and regulate prices for the transportation of freight, to negotiate contracts and agreements with individuals and other companies, and to see that such contracts are fairly and equitably complied with: also to investigate and examine all claims for damages, and losses of freight or baggage, and to certify such of them as are found valid to the General Superintendent for approval.

The General Ticket Agent is required, with the approval of the President or General Superintendent, to regulate the prices for the transportation of passengers, to negotiate ticket arrangements with other companies, and to supervise all matters connected with the sale of tickets.

The General Fuel Agent is required to provide the necessary supply of fuel for use on the road, to have it delivered in such quantities and of such qualities as shall be in accordance with the specifications, to see that it is carefully measured before payment is made; and to submit all contracts to the General Superintendent for approval. At the end of each month it is his duty to furnish an accurate statement of all the fuel consumed during the month, and at the end of each quarter he is required to make a correct inventory of the amount received during the quarter, the sum paid for the same, the quantity consumed, and the fuel on hand, stating the locality of the same.

The Superintendent of Telegraph is held responsible for the proper working and economy of the line, and prompt transmission of communications. He is required to perfect such arrangements as will enable Superintendents of Divisions, and other officers of the company, to avail themselves to the fullest extent of the advantages to be derived from the use of the telegraph in

communicating information or directions in regard to the movement of trains, or business operations of the road, and to see that the instruments, batteries, and other property in his charge are kept in good order and preservation.

The Foreman of Bridge Repairs has general charge of all the bridges upon the road, is required to examine them frequently, and is held responsible for their safety.

"The enforcement of a rigid system of discipline," says Mr. McCallum, "in the government of works of great magnitude, is indispensable to success. All employees should be accountable to, and be directed by, their immediate superiors only; as obedience cannot be enforced when the foreman in immediate charge is interfered with by a superior officer giving orders directly to his subordinates. It is very important, however, that principal officers should be in possession of all the information necessary to enable them to judge correctly as to the industry and efficiency of the employees of any grade. To acquaint themselves in this particular, and remedy imperfections without weakening the influence of subordinate officers, should be the aim of the heads of departments. Each official should possess all the power necessary to render his position efficient, and have authority, with the approval of the President or General Superintendent, to appoint all persons for whose acts he is held responsible, and to dismiss any subordinate when in his judgment the interests of the company will be promoted thereby."

System in Railway Reports.

The following extracts regarding systems of reports and checks are from Mr. McCallum's report: —

"Hourly reports are received by telegraph, giving the position of all the passenger and the principal freight trains. In all cases where passenger trains are more than ten minutes, or freight trains more than half an hour, behind time, on their arrival at a station the conductors are required to report the cause to the

operator, who transmits the same by telegraph to the General Superintendent; and the information being entered, as fast as received, in a convenient tabular form, shows at a glance the position and progress of trains, in both directions, on every division of the road.

"The importance of ascertaining the particulars connected with delays cannot be over-estimated, as they are frequently the result of mismanagement, and often the primary cause of accidents, and in their history is developed a class of facts and delinquencies which could not be so easily detected in any other way. By these means the prevailing causes of delays are made known, and an opportunity is given to apply the corrective, where the nature of the case will permit.

"The daily reports of passenger conductors give the designating numbers of the engines and cars, the names of the persons employed on the trains, the time of arrival at and departure from the several stations, the particulars in regard to delays, and such other matters of interest as occur on the trip.

"The daily reports of freight conductors, in addition to the above, give a general description of the load contained in each car, the place whence taken, where left, and destination as per way-bills.

"The station agents' daily reports give the time of the arrivals at and departures of all freight trains from their respective stations; the name of the conductor and number of the engine; the numbers of the cars taken and left, with the tonnage of freight in each; the numbers and kinds of cars remaining at their station, and whether the same be loaded or empty; how many are required for the business of the station, and the nature of that business; and whether any conductor has refused to take cars or freight way-billed to other stations; also a statement of freight over or short of bills, or damaged or wrongly directed; delays of freight, and causes thereof; damaged cars at stations, with particulars as to cause of the same; an accurate daily report of all baggage received, forwarded, and remaining on hand, and any

other information of interest pertaining to the business of their respective stations.

"The division superintendents report monthly the number of miles run, the expense of engineers and firemen, the quantity and cost of oil, waste, and tallow for each engine on their respective divisions.

"The masters of engine repairs report monthly the amount expended for repairs on each engine.

"The general wood agent reports monthly the number of cords of wood used by each engine, and the cost of the same."

From the information thus obtained, properly arranged at headquarters, the following details for the different trains on the several divisions are deduced:—

The speed of the train between the several stations.
The average load carried in each car.
The tonnage of useful load carried.
The tonnage of cars in which it was transported.
The tonnage of empty or returned cars.
The position or location of the cars.

For each locomotive the following record is obtained:—

The number of miles run.
The cost per mile run for engineer and fireman.
The gallons of oil used.
The miles run to one pint.
The pounds of waste used.
The pounds of tallow used.
The cost for oil, waste, and tallow.
The cost for repairs.
The cost per mile run for repairs.
The cords of fuel used.
The cost of fuel and the cost per mile run.
The total cost for all of the above items.
The cost per ton per mile for the same.

The above report enables an exact determination to be made of the speed of trains, of the ratio between the useful and the dead load hauled, and of the relative work done by, and economy of, the different engines. The progress of every car is traced over the road ; nor can one remain at any station an unnecessary time, for the purpose of being unloaded or forwarded, without exciting attention. Thus the movement of every ton of freight over the road is within the knowledge and control of the managing department.

The Pennsylvania Railroad.

In the transportation department of the Pennsylvania Railroad the General Superintendent has charge of all property belonging to the company ; is responsible for the movement of all engines and trains, and the safe transportation of persons and property ; orders tools and materials for repair, and increase of road equipment and appointments ; makes special rates for local freights, furnishes current supplies by order, and appoints employees and agents of his office and on the lines ; and all this with the consent of the President. In the performance of these duties he has three assistants : The Assistant Superintendent proper, in special charge of the Rolling Stock, responsible for the distribution, the numbering, marks of identification, weight and record thereof, and acting, on occasion, in place of the Superintendent. The Chief Engineer of Maintenance of Way, in charge of repairs of track, roadway, bridges, and buildings. The Superintendent of Motive Power and Machinery, in charge of the maintenance of engines and rolling stock, and the shops and employees connected therewith, assisted by a mechanical engineer, by whom designs for all work, both of repairs and manufacture, in all the shops of the company and in private manufactories, are prepared.

The main line of the Pennsylvania Railroad is divided into three divisions, 106, 131, and 117 miles in length, respectively. Each division has its own employees throughout, who never run

over another division, the cars at a terminal point passing into the hands of the Despatcher of the next division, who makes them up into trains of the proper kind and number, and puts them in charge of the proper persons, and starts them on their way, the train-master assigning engines, engine-men, and train-men.

The Supply Department is in charge of a Purchasing Agent, who keeps on hand, at the storehouse in Philadelphia, all articles in general use in every department, under the Superintendent's charge (deliverable to his order), except fuel, and materials for roadway and bridges.

The Engineer Department is managed by a Chief Engineer of Construction, or a Consulting Engineer, having charge of the original maps and profiles, plans of real estate and structures, and the record of expenditures for these purposes, and also the preparing of all plans, estimates, and specifications for new work. The Chief Engineer of Maintenance of Way has exclusive charge of all repairs of road, buildings and works, and the fuel and water supply. Assisting him are Resident Engineers, in charge of 100 miles, more or less. Under each Resident Engineer are Supervisors, each in charge of about 40 miles, and beneath these latter track foremen, with their gangs, each in charge of from 2 to 4 miles of double track. These foremen are required to go over their section daily, according to specific instructions; and there are, besides, track watchmen, night and day, at exposed points, such as side cuts, wooden bridges, etc.; while in winter, and severe nights in spring and fall, a force is employed sufficient to enable a watchman to pass over every portion of the track before any passenger train is due.

A very essential point, and one to which especial attention is paid upon the Pennsylvania Railroad, is the educating of its officers and men in the particular field in which they are to be employed. This point is worth far more consideration from railway managers than it has commonly received. Upon the Pennsylvania Railroad dwelling-houses have been furnished to the general officers at Altoona, to the Resident Engineers, Division

Foremen, Despatchers, Supervisors, and other local officers, and the company have done every thing to cultivate local attachments among its employees. A system of co-operation between the company and its workmen has also been adopted, by which great economy has been promoted. Premiums have been paid to all foremen who exert themselves in an unusual degree in the direction of economy, both of material and labor. So, too, in the running department, the value of all fuel and stores per mile run saved over a certain established quantity, is equally divided between the company and the employees, engineers receiving 65 per cent., firemen 30 per cent., and others engaged in handling stores 5 per cent. The saving in a single year, by this system, amounted to $100,000.*

The Use of Railway Reports.

The reports of railway companies in this country do not, as a general thing, furnish the means of analyzing the cost of opera-

* The apprentice system upon the Pennsylvania Railroad is worthy of notice. Apprentices are taken at 17 years of age, if possible, on their birthday, and bound for four years, receiving 75 cents a day the first and second years, 90 cents the third, and $1.00 the fourth. If during their apprenticeship their conduct and service has been satisfactory, they receive $124 bonus at the end of the four years, and afterwards they have, as skilled workmen, from $1.50 to $2.90 per day. Work commences in the locomotive shop with vice-filing, from which the apprentice passes to machine work, running tools, and setting up engines. In carpentry, freight is followed by passenger car work. By this method not only does the workman become skilful in the performance of the different operations involved in engine and car building, but ideas of order and discipline are acquired which fit him afterwards to take charge of work. This method has proved entirely satisfactory in every respect. There is no better example of a complete organization for the maintaining and operating of a large railway than that of the great road above named. An examination of the whole route from Philadelphia to Pittsburg, with the extensive shops, the equipment, and the system in use at the various offices, is a most useful and satisfactory study for the railway engineer and the railway manager. Everything is done upon this road to induce uniformity in the design and execution of all work, gradually bringing all machinery and equipment to a common, thoroughly tested, and approved standard.

tion. The gross expense is given, and to some extent it is divided among the several departments of Roadbed, Machinery, and Transportation; but there are no data given from which we may arrive at a satisfactory solution of the numerous important problems which present themselves in the working of a railway. Besides the retrospective use of a minute division of expenses, there is a prospective one, viz., the formation of estimates for future operations, and a basis for the establishment of tariffs. If the conditions of the traffic remain the same, an estimate of the cost of future operation may be made with a good degree of certainty; but if the conditions affecting the traffic change, as they do from time to time, the data for estimates change also. That we may at all times possess these data, we should know every year just the cost of transporting the several classes of freight. It is not enough that the gross income exceeds the whole expense; even then the road may be working at a disadvantage. Unless each item of transport pays for itself, the tax upon some other item must pay for it.

It is, of course, the object to make the gross receipts exceed the total expense by the largest possible amount. Thus the net income depends no less upon the reduction of the working expenses, than upon increasing the direct revenue. The gross receipts are governed by the charge per mile per unit transported, by the number of units transported and by the distance they are moved. Of these elements the company controls the first only, except so far as the adjustment of tariffs may attract business. Reduction of rates will often increase the number of units transported, but this reduction may be carried too far. We should endeavor to work the road and its equipment as nearly as may be to its full capacity, since then the charge for each unit transported on account of the interest on capital will be the least possible. A large part of the expense of maintenance also is independent of the amount of work done, and thus the larger the amount of traffic, to a certain extent, the less the cost of transport per unit of material transported. Other parts of the ex-

penses depend directly upon the amount of work done. The adopting of low rates in order to fill return trains which would otherwise run empty, or in order to build up a particular kind of traffic, or to develop any district which will eventually furnish a profitable tribute to the road, is good economy.

Relation between Revenue and Expenses.

The general relation between the revenue and the expenses may be seen by the following figures:—

Upon the roads of Ohio, in 1870, the total earnings were \$52,895,812; the operating expenses \$37,020,331; and the net income \$15,875,638; making the expenses 70 per cent. of the gross earnings. Upon the roads of New York, for 1868, the earnings were \$49,377,790; the expenses \$35,737,830; the net income \$13,639,960; and the percentage of expense to gross income 72 very nearly. Upon the roads of Massachusetts, in 1869, the gross income was \$24,539,722; the working expenses \$17,342,992; the net income \$7,196,730; and the per cent. of expense to income 70 very nearly. Upon the Pennsylvania Railroad, for 1869, the earnings were \$17,250,811; the expenses \$12,203,267; the net income \$5,047,544; and the percentage of expense to income 71 nearly. Upon the Baltimore and Ohio Railroad, for three years, the results were as follows:—

Earnings and Expenses.	1868.	1869.	1870.
Earnings, ...	\$7,558,644	\$8,724,915	\$8,427,728
Expenses, ...	5,054,448	5,756,106	5,453,460
Net income, ...	2,504,195	2,968,809	2,974,268
Per cent. of expense to inc.,	66.86	65.97	64.70

The proportional receipts from passengers, freight, and other sources, are shown by the following figures:—

REVENUE.	Railways of New York for 1868, 3054 Miles.	Railways of Mass. for 186[illegible], 1241 Miles.	Railways of Ohio for 1870, 2889 Miles.
Revenue from passengers, .	$14,855,689	$10,296,175	$16,802,719
Revenue from freight, . . .	31,570,968	10,966,450	33,348,195
Revenue from other sources,	2,951,133	1,232,745	2,744,898
Total income, ...	$49,377,790	$22,495,370	$52,895,812

Upon the Pennsylvania and the Baltimore and Ohio Railroads, the proportions between the passenger and freight revenue for the year 1869, were as follows: —

REVENUE	Pennsylvania Railroad.	Baltimore and Ohio Railroad.
Revenue from passengers, ...	$ 3,500,071	$1,677,862
Revenue from freight, ...	12,932,656	7,999,011
Revenue from other sources, . .	818,084	. . .
Total income, ...	$17,250,811	$9,676,873

From these figures it will be seen that by far the larger part of the revenue comes from freight, except in the case of the roads of Massachusetts, which consist of a great number of short lines traversing a thickly settled country, where the two revenues are about equal.

Division of Receipts.

With regard to the relative receipts from local and from through business the published reports furnish but little informa-

tion. By far the larger part, however, of the revenue comes from local work. The average distance travelled by passengers upon the New York railways, in 1868, was 35.7 miles. The average number of miles by each ton of freight was 109.39. The average number of miles travelled by each passenger in Ohio, in 1870, was 45. The number of tons of through freight carried was 530,526; of local freight, 1,613,198. The average distance travelled by passengers upon the Pennsylvania Railroad, in 1868, was 35.5 miles. Of 4,427,884 tons of freight transported, 3,818,994 tons were local, and 608,890 tons through; the mileage of the local freight being 457,793,285, and that of the through freight 217,982,275. The revenue from local freight was $8,843,002, and from through freight $4,039,163. In other words, upon this great through route over two thirds of the revenue from freight came from local traffic; the number of tons of through freight being only 14 per cent. of the whole amount hauled. So, too, while the whole number of passengers carried was 3,747,178, the equivalent number of through passengers would be only 375,207. The same result, very nearly, appears for the following year. The general result of operation per mile of road (main line and branches) in Massachusetts, New York, and Ohio, has been as below: —

EARNINGS AND EXPENSES.	New York, 1868	Mass., 1869.	Ohio, 1870.
Passenger earnings, ...	$3,954	$5,148	$3,008
Freight earnings, ...	8,403	5,483	5,308
Total revenue, ...	13,143	11,247	8,419
Operating expenses, ...	9,533	8,019	5,892
Net income, ...	3,610	3,228	2,527

The total revenues above exceed, by a small amount, the sum of the passenger and freight revenues, as the total includes certain earnings not returned under either passengers or freight.

The Amount of Work done upon Railways.

With regard to the amount of work done per mile of road, we have also the following :—

Mileage and Tonnage.	New York, 1868.	Mass., 1869.
Miles run by passenger trains, . .	2,484	2,771
Passenger mileage, ...	175,161	204,873
Number of passengers carried, . .	4,908	13,367
Miles run by freight trains, . . .	3,300	2,641
Tonnage mileage, ...	348,270	160,730
Number of tons carried, ...	3,184	3,545
Total miles run, ...	5,783	5,691

The amount of work done upon the roads of New York State for the year 1868, as given in the annual report of the State Engineer, is shown in the table below :—

Length of main lines, miles, ...	3,054
Length of branches, ...	703
Double track and sidings, ...	1,397
Length of equivalent single track, ...	5,154
Miles run by passenger trains, ...	9,329,671
Miles run by freight trains, ...	12,396,129
Miles run by freight and passenger trains, .	21,725,800
Number of passengers carried, ...	18,434,300
Passengers carried one mile, ...	658,078,513
Tons of freight carried, ...	11,961,692
Tons carried one mile, ...	1,308,451,978
Income from passengers, ...	$14,855,689

Income from freight, ...	$31,570,967
Average passengers per train, ...	70.81
Average miles travelled per passenger, . . .	35.70
Average tons per freight train, ...	105.82
Average miles each ton was carried, ...	109.39

From the returns made by the several railroad corporations in Massachusetts to the State, for the year 1869, we have the following: —

Miles run by passenger trains, ...	5,542,364
Miles run by freight trains, ...	5,283,238
Number of passengers carried, ...	26,735,105
Passenger mileage, ...	409,746,975
Receipts from passengers, ...	$10,296,175
Tons of freight carried, ...	7,091,443
Tonnage mileage, ...	321,461,246
Receipts from freight, ...	$10,966,450
Total income, ...	$22,495,370

Division of the Expense of Operation.

The general division of the expense of operation, the expense per mile of road, and the expense per mile run for the railways of New York during 1868, was as below: —

DIVISION OF EXPENSE.	Total Expense.	Expense per Mile of Road	Expense per Mile Run.
Maintenance of way, . .	$13,074.594	$3,480	$60.18
Machinery and cars, . .	7.491.851	1.994	34.50
Transportation, ...	15.250.716	4.059	70.20
Total, ...	$35.817.161	$9,533	$164.88

The division of the expense of operation separated between the passenger and the freight departments is given below : —

HEADS OF ACCOUNTS.	Allotted to Pasenger Transportation.	Allotted to Freight Transportation.
Repairs of road, exclusive of iron, . . .	$2.066.848	$3.319.316
Cost of iron for repairs.	1,493.307	2.787.561
Repairs of buildings, ...	373.979	601,816
Repairs of fences and gates, ...	47.981	93.418
Taxes on real estate, ...	474 342	759.504
Total for maintenance of roadway, . .	$4.456,459	$7.561,617
Repairs of engines, ...	1,087,519	1.853.049
Repairs of cars, ...	1.214.401	2,245.722
Repairs of tools, ...	82.317	160.640
Incidentals, oil, fuel, etc., ...	111,259	210.455
Total for repairs of machinery, ...	$2.495.498	$4.469.868
Office expenses, stationery, etc., ...	99.650	213.703
Agents and clerks, ...	545.364	1,051,224
Labor loading and unloading freight, . .	64.899	1.218,964
Porters, watchmen, and switchmen, . . .	348,928	566,428
Wood and water station attendance, . .	58.745	98.242
Conductors, baggagemen, and brakemen, .	697.106	960,912
Enginemen and firemen, ...	616,619	1,115,939
Fuel, cost and labor of preparing for use, .	1.510.246	2.331.778
Oil and waste for engines and tenders, . .	165,913	265.509
Oil and waste for cars, ...	27,890	49.087
Loss and damage of goods and baggage, .	11.575	257.575
Damages for injuries to persons, ...	528.310	16.586
Damages for property and for cattle killed,	14.462	27.253
General superintendence, ...	137.510	222.381
Contingencies, ...	594.351	962.830
Total transportation expenses, ...	$5.421.575	$9.358.417
Grand total, ...	$12.373.532	* $21,389.902

* Besides the items above, there are $2.053,724 of expenses not divided between passenger and freight transportation, of which $1,056,517 is for main-

The division of expenses between the passenger and freight departments is seldom made in the reports of railway companies, nor are the data given by which it can be done. It is doubtful, in most cases, if the companies themselves would be able to do it. The method adopted by Mr. McAlpine for making this division is as follows: —

"Ascertain the whole weight (in tons carried one mile) of the trains employed in passenger transportation, and also for freight, including in this the weight of the engine, tender, an average load of wood and water, of the baggage, mail, express, and passenger cars, or of the freight cars, as the case may be, and also the weight of the passengers and their baggage, and of the freight.

"If both classes of trains have been moved at the same rate of speed, the division of the expenses between the passenger and the freight business will be in the proportion of the weights of each class of trains, as above ascertained, moved one mile.

"If the speed of the passenger trains is double that of the freight trains, then the weight of the former must be increased by 40 per cent., and the division made in the proportion of this increased weight to that of the freight trains.

"If the speed of the passenger trains is two and a half times that of the freight trains, the weight of the former must be increased by 50 per cent., and the division made as before stated.

"If the speed of the passenger trains is three times that of the freight trains, the weight of the former must be increased by 60 per cent., and the division made as before stated."

The division of expenses upon the New York and Erie Road for the years 1854 and 1855, as allotted to passengers and to freight, is given on page 442: —

tenance of roadway, $526.484 for repairs of machinery, and $470.723 for transportation. In making the above extracts from the State Engineer's report the cents have been omitted, which will account for the slight discrepancy which would be found in the totals if the separate items were added up.

DISTRIBUTION OF ACCOUNT	Cost per Passenger per Mile. 1854.	1855.	Cost per Ton per Mile. 1854.	1855.
	cts.	cts.	cts.	cts.
Office and Station Expenses.				
Office expenses and stationery,	.018	.025	.022	.022
Agents and clerks,	.044	.062	.056	.051
Labor, loading and unloading,	. .	.	.097	.097
Cost of Running.				
Porters, watchmen, and switchmen,	.019	.026	.024	.023
Wood and water station attendance,	.002	.004	.002	.003
Fuel, cost and labor of preparing,	.143	.157	.208	.205
Passenger conductors, baggagemen, and brakemen,	.079	.097	. .	. .
Freight conductors and brakemen,	. .	. .	.100	.092
Passenger enginemen and firemen,	.096	.107	. .	. .
Freight enginemen and firemen,	. .	. .	.099	.080
Oil and waste for passenger trains,	.038	.041	. .	. .
Oil and waste for freight trains,	. .	. .	.040	.040
General Expenses.				
Loss and damage, passenger and freight,	.019	.005	.024	.014
General superintendence,	.022	.028	.028	.023
Contingencies,	.035	.057	.044	.048
Repairs of Engines and Cars.				
Engines and tenders, passenger,	.137	.109	. .	. .
Engines and tenders, freight,	. .	. .	.140	.081
Passenger and baggage cars,	.082	.079	. .	. .
Freight cars,	. .	. .	.094	.045
Tools and machinery in shops,	.010	.008	.010	.006
Incidental expenses about shops,	.009	.010	.010	.009
Repairs of Track and Roadway.				
Road bed,	.020	.022	.024	.018
Track,	.168	.207	.212	.161
Fences, gates, etc.,	.002	.004	.003	.003
Repairs of Structures.				
Truss bridges,	.008	.009	.010	.009
Wood and water stations,	.006	.009	.007	.008
Engine and car houses and shops,	.001	.002	.002	.002
Miscellaneous.				
Superintendence and office expenses,	.001	. .	.002	. .
Taxes,	.025	.027	.030	.022
Contingencies,	.003	.006	.005	.005
Ferry,	. .	. .	.097	.075
Expenses operating telegraph,	.012	.014	.016	.013
Total,	.999	1.118	1.406	1.155

It has been thought best to introduce the foregoing table, for though it relates to the expense of operation sixteen years ago, it is one of the few cases in which the passenger and freight expenses are separated.

Expenses of Operation upon the Pennsylvania Railroad.

The reports of the Pennsylvania Railroad Company contain a large amount of valuable information concerning the details of operation. From the report of the operations for the year 1869, we obtain the following: —

Miles run by passenger trains,	2,302,968
Miles run by freight trains,	6,904,888
Miles run by other trains,	366,776
Total miles run,	9,574,632
Passengers carried,	4,229,363
Passenger mileage,	144,728,652
Income from passengers,	$3,631,137
Average miles travelled per passenger,	34.22
Average income per passenger, cts.,	85.85
Income per passenger per mile, cts.,	2.51
Tons of freight transported,	5,402,991
Tonnage mileage,	752,711,312
Income from freight,	$12,932,656
Average distance per ton, miles,	139.31
Average income per ton, cts.,	239.36
Income per ton per mile, cts.,	1.72
Total income from all sources,	$17,250,811
Income per mile run, cts.,	1.80
Total cost of operation,	$12,203,267
Cost of operation per mile run, cts.,	1.27
Net income,	$5,047,544
Net income per mile run, cts.,	.53
Cost of maintenance of way per mile run, cts.,	34.9
Cost of motive power per mile run, cts.,	38.4
Maintenance of cars per mile run, cts.,	15.3
Transportation expenses per mile run, cts.,	36.6

The following shows the division of expenses between the passenger and the freight departments upon the Pennsylvania Railroad for 1869: —

HEADS OF ACCOUNT.	Passenger Department.	Freight Department.	Total Expense.
Advertising, ...	$25.592.12	$7.086.79	$32.678 91
Agents, ...	38.699.37	60.481.03	99.180.40
Attendants, ...	1.266.58	3.799.72	5.066.30
Baggage masters, ...	46.159.35	. . . .	46.159.35
Ballast, ...	29.910.67	89.732 05	119.642.72
Brakemen, ...	67.183.16	493.538.21	560.721.37
Bridges, repairs of, ...	123.041.52	369.124.53	492.166.05
Car furniture and fixtures, ...	23.144.34	37.141.78	60 286.12
Car shops and sheds, repairs of, . .	4.756.48	14.269.61	19.026.09
Car service, ...	18.093.52	18.150.05	36.243.57
Cars, cleaning and inspecting, . . .	63 790.41	67.472.86	131.263.27
Cars, repairs of, ballast and wood, .	1.142.41	3.427.36	4.569.77
Cars, repairs of freight, ...	. . .	961.286.24	961.286.24
Cars, repairs of pass. and baggage, .	450.394.92	. . . .	450.394.92
Cars, road and hand, ...	7.483.62	22.450.88	29.934.50
Chairs, ...	32.781.34	98.344.07	131.125.41
Clerks, ...	58.240.88	223 849.23	282.090.11
Coal for engines, ...	92.678.69	505.450.76	598.129.45
Conductors, ...	63.195.33	184.210.58	247.405.91
Cotton waste, ...	4.862.31	17.057.80	21.920.11
Cross ties, ...	66.170.30	198.510.86	264.681.16
Despatchers, ...	19.978.90	62.352.87	82.331.77
Drawbacks and overcharges, ...	2.679 99	154.406.73	157.086.72
Engineers and firemen, ...	113.626.85	448.909.78	562.536.63
Engine house and machine shop rep..	98.261.85	294.785.68	393.047.53
Expenses of stations except labor, .	6.607.41	39 644.58	46.251.99
Expenses of grain elevator, ...	. . .	382.33	382.33
Expenses on property, ...	8.258.39	24.775.25	33.033.64
Foreign agencies, ...	129.910.41	62 957.76	192.868.17
Foremen and watch houses, reps. of,	10.565.92	31.697.72	42.263.64
Frogs, ...	9.337.16	28.011.53	37.348.69
Fuel and light, ...	292.81	878.48	1.171.29
Fuel at stations, ...	6.494.94	2.648.36	9.143.30
Fuel for cars, ...	8.019.96	65.50	8 085.46
Fuel for engine houses and shops, .	3 827 04	11.481.18	15.308.22
Incidentals, ...	56.196.10	161.068.46	217.264.56
Iron rails, ...	192.031.79	576.095.43	768.127.22

HEADS OF ACCOUNT	Passenger Department.	Freight Department.	Total Expense
Labor at stations.	$27.134.07	$152,883.98	$180.018.05
Laborers.	56.941.61	170.824.05	227.766.56
Light at stations.	14.821.43	10.504.12	25.325 55
Light for cars.	8.493.10	5.405.41	13.898.51
Locomotive furniture and fixtures. .	7.986.15	23.958 63	31.944.78
Locomotives. repairs of.	293.187.04	991.561.85	1,284.748.89
Loss and damage.	. . . .	56.370.66	56.370.66
Loss from injuries to individuals. .	24.157.92	. . . .	24.157.92
Mail expenses.	4.233.97	. . . .	4.233.97
Office expenses.	2.724.88	8,174.65	10.899 53
Oil.	11.684.22	48.057.89	59.742.11
Oil. tallow. sponge. wool. etc., . .	15.634.11	55.450.17	71.084.28
Real estate in Philadelphia.	274.27	822.75	1.097.02
Road bed repairs. labor.	68.975.26	206.925.75	275.901 01
Road bed repairs. materials. . . .	34.324.86	102.974.58	137.299.44
Salaries of general officers, ...	17.066.61	51.199.99	68.266.60
Snow and ice. removing.	5.074.13	15.222.41	20.296.54
Spikes.	7.289.57	21.868.72	29.158.29
Stationery and printing.	29 987.72	68.079.23	98.066.95
Stations. reps. of. rent and furniture.	74.356.88	108.474.22	182.831.10
Superintendence and supervisors. .	15.760.09	47.220.23	62.980.32
Switches.	6.517.21	19 551.59	26.068.80
Switchmen.	13.047.92	39.143.91	52,191.83
Tallow.	5.792.37	27.898.41	33.690.78
Tax on depots.	9.612.82	28.862.12	38.474.94
Tax. United States excise.	94.970 44	4.322.41	99.292.85
Tax. tonnage (State	. . . .	68.965.13	68.965.13
Tax. State revenue.	30 460 65	95.876.20	126.336.85
Taxes on engine houses and shops. .	5.948.07	17.844 25	23.792.32
Taxes on real estate for road. . . .	5.141.66	15.425 08	20.566.74
Teaming.	596.20	109.125.02	109.721.22
Telegraph expenses.	22.946.01	68.438.90	91.384 91
Telegraph. repairs of.	6.279.14	18 837.46	25.116.60
Tools and machinery. repairs of. . .	55.986.88	167.961.02	223.947.90
Tolls to other roads.	85.758.23	242.297.85	328.056 08
Track. labor repairing.	151.271.72	453.815.05	605.086.77
Watchmen.	37.632.83	112.898.63	150.531.46
Water and coal stations. repairs of. .	5.487.51	16.462.72	21.950.23
Water and coal stations. expenses of.	7.940.63	23.822.06	31.762.69
Wood. and labor preparing.	18.061.09	81.957.44	100 018.53
Total expenses.	$3.168.236.13	$9.035.031.47	$12.203.267.60

The items on pages 444 and 445 may be recapitulated under the several general heads of expense as follows: —

HEADS OF ACCOUNTS.	Passenger.	Freight.	Totals.
Conducting transportation. .	$1.021.197.43	$2.482.595.14	$3.503.792.57
Motive power.	794.494.28	2.884.700.87	3.679.195.15
Maintenance of cars.	463.689.27	1.001.169.95	1.464.859.22
Maintenance of way.	835.392.03	2.506,176.07	3.341.568.10
General expenses.	53.463.12	160.389.44	213.852.56
Totals.	$3.168.236.13	$9.035.031.47	$12.203.267.60

Deductions from the Preceding Figures.

From the preceding tables we may deduce the cost per mile run, or the cost per ton or passenger per mile, for the whole or any part of the working expenses. Such deductions are useful as a means of comparing the cost of operation from year to year under varying conditions of traffic and different systems of management, and also as a means of obtaining something like a standard for railway economy. For example, we may deduce the following details from the tables given above: —

SPECIFICATION.	Pennsylvania Railroad. 1869.	Roads of Massachusets. 1869.	Roads of New York. 1868.
Passengers carried per mile run by pass. trains.	1.84	4.82	1.98
Tons carried per mile run by freight trains. .	0.78	1.34	0.96
Average miles travelled per passenger.	34.22	15.33	35.70
Average miles each ton was carried.	139.31	45.33	109.39
Income per passenger carried. cts..	85.85	38.51	80.60
Income per passenger per mile. cts..	2.51	2.51	2.26
Income per ton of freight carried. cts..	239 36	154.65	263.93
Income per ton per mile. cts..	1.72	3.41	2.41

It will be seen that while the income per passenger per mile is more in Massachusetts than in New York, the income per passenger carried is much less, and the same with regard to freight. This arises from the fact that the average distance travelled by passengers in Massachusetts is only 15.33 miles against 35.70 in New York; and that the average distance for a ton of freight in Massachusetts is only 45.33 miles against 109.39 in New York.

The Massachusetts returns do not enable us to determine the cost of transport for passengers and freight separately, but the reports of the New York roads and of the Pennsylvania Railroad do, from which we obtain the following: —

MILEAGE, INCOME, AND EXPENSE.	Pennsylvania Railroad. 1869.	New York Roads. 1868.
Income per passenger per mile, cts.,	2.51	2.26
Expense per passenger per mile, cts.,	2.19	1.88
Net income per passenger per mile, cts.,	0.32	0.38
Income per ton per mile, cts.,	1.72	2.41
Expense per ton per mile, cts.,	1.20	1.63
Net income per ton per mile, cts.,	0.52	0.78

In view of the above facts, the statements of certain recent writers that freight may be transported for half a cent per ton per mile, and that passengers may be carried from New York to Philadelphia for ten cents, and from Boston to New York for fifteen cents, will hardly be accepted. A common error in the estimates of cheap transportationists, is the assuming the trains to be always filled. One of the public railway economists bases his estimate upon 2000 passengers per train: while the average number in New York, for 1868, was 71 only. So, too, another writer reckons that each freight train will carry 300 net tons, while upon the roads of New York, for 1868, the average number of tons per freight train was only 106. Another item, entirely ignored by the advocates of cheap transportation, is the interest to be paid upon the capital expended in the construction of the railway.

Fares for Long and Short Distances.

Public attention has been called of late years to the relation that exists between the charges upon railways for short and for long distances; and an attempt has been made to compel railway companies to charge no more per ton or per passenger per mile for local than for through traffic. However judicious it may be, in certain cases, to build up a local traffic by a special reduction of rates, the general principle of charging a higher rate for short than for long distances, or, to state it more properly, of charging a less rate for long than for short distances, is correct, since the cost of transport is not in the ratio of the distance. A portion of the expense of transportation is independent of the distance; whence, for that portion, the greater the distance the less the cost, or the charge per mile. The table below was deduced by Dr. Lardner, from the returns of the Belgian railways for 1844. Each ton of freight was carried an average distance of 45 miles, and produced an average receipt of 70 d. The expenses were for each ton, 51 d. of which 34 d. were independent of the distance, and 17 d. dependent upon it.

The figures are given in pence and decimals.

Cost of Transport per Mile	Expenses per Ton carried independent of Distance.	Tariff per Ton per Mile.	Average Distance carried.	Total Receipts per Ton carried.	Total Expenses per Ton carried	Total Expenses per Ton per Mile.
0.377	34.0	1.55	45	69.7	50.9	1.13
		1.43	50	71.5	52.8	1.05
		1.33	55	73.2	54.7	0.99
		1.25	60	75.0	56.6	0.94
		1.19	65	77.5	58.5	0.90
		1.13	70	79.1	60.4	0.86
		1.08	75	81.0	62.3	0.83
		1.04	80	83.2	64.2	0.80
		1.00	85	85.0	66.1	0.78
		0.96	90	86.4	68.9	0.76
		0.93	95	88.3	69.8	0.74
		0.92	100	92.0	71.7	0.71

In the fourth column a series of increasing distances are given. In the third column are exhibited the corresponding values of the tariff. In the fifth column is given the total receipts which would be obtained for each ton carried; and in the sixth column the total expenses. In the seventh column is given the expense per ton per mile.

By this last column is rendered apparent the increased saving per mile on the expenses of transport produced by the augmented average distance.

From this table it appears that if the average distance to which each ton of goods is transported were doubled, a tariff fifty per cent. less would yield the same amount of profit per ton carried; and if a less reduction of the tariff would produce this augmented distance, an increased profit would arise, both from the increased quantity of goods carried and from the increased average distance.

"Thus," says Dr. Lardner, "it is evident that consistently with realizing the same proportion of profits upon the business executed, a railway company can always afford to reduce the charge per mile in a greater or less proportion as the distance increases."

It is, however, not to be denied that in many cases the tariffs for local work have been made by the companies unwarrantably large, thus producing a very bad effect upon the industries of way towns. In many cases companies have been driven by competition to charging rates so low for long transport as to work such traffic at a loss, and the local charges have been made large enough to cover this loss. Such management is bad, both for way towns and for the road; but even this fault arises, in many cases, from ignorance on the part of the company of the actual expense of doing work. So little attention has as yet been paid in this country to the *science* of railway transport that even the data for solving the important problems relating to the establishment of tariffs are almost entirely wanting.

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AMOUNT OF EQUIPMENT UPON RAILROADS.

The amount of equipment employed upon railways is shown by the following figures, taken from the report of the State Engineer of New York, for 1868, and from the report of the Railroad Commissioner of Ohio, for 1870:—

	New York.	Ohio
Length of main track,	3,054	2,889
Length of branches,	703	484
Double track and sidings,	1,397	637
Equivalent single track,	5,154	4,010
Number of locomotives,	1,111	1,572
Number of passenger cars,	1,163	901
Number of freight cars,	17,934	26,440
Baggage, mail, and express cars,	362	401

From the above it appears that in New York each mile of road (main track and branches) requires 0.296 of a locomotive, 0.309 of a passenger car, and 4.8 freight cars. In Ohio each mile requires 0.466 of a locomotive, 0.266 of a passenger car, and 7.87 freight cars. In other words, in New York we require—

One locomotive for 3.38 miles of road;
One passenger car for 3.23 miles of road;
One freight car for 0.21 miles of road.

And in Ohio—

One locomotive for 2.15 miles of road;
One passenger car for 3.74 miles of road;
One freight car for 0.13 miles of road.

SERVICE OBTAINED FROM THE EQUIPMENT.

With regard to the amount of service obtained from the equipment, we have the following figures from the reports above re-

ferred to. The 1111 engines in New York ran 9,329,671 miles with passenger trains, and 12,396,129 miles with freight trains, or 21,725,800 miles in all; or 19,555 miles per engine, or 62.5 miles a day (313 days in a year). The 1572 engines in Ohio ran 39,170,297 miles, or 24,918 miles each, or 79.6 miles a day. From 3 to 5 hours a day may thus be called the effective running service of a locomotive; though an engine will be standing with steam up as many hours as it is running. The actual service of an engine while upon the road is, of course, greater than above stated, as the figures include all engines out of service while under repairs. Upon the Pennsylvania Railroad, in 1869, the general average passenger engine mileage was 24,763, and that of freight engines 21,052. The cost for repairs per 100 miles run for the same time was $11.00; for fuel, $7.29; and for stores, $1.20. The pounds of coal consumed per mile run averaged 62.90. The quarts of oil used per 100 miles were 2.90; pounds of waste, 1.90; pounds of tallow, 2.80. Upon the Illinois Central Railroad, for the same year, the miles run per ton of coal averaged 37.04; miles run per pint of oil, 14.37; the cost in cents per mile run for repairs, was 11.27; for fuel, 6.56; for cleaning, 1.05; and for oil and waste, 0.73. The average run per cord of wood, in the Northern States, has been 25 miles; though double this distance has been accomplished upon special occasions.

Paying and Non-paying Load.

No point in railway management demands a more vigorous attempt at reform than the reduction of the dead weight hauled. The relation between the paying and the non-paying load transported on the English railways, in 1867, is shown by the following figures from the Board of Trade returns: — *

* Paper read before the Civil and Mechanical Engineers' Society, by Mr. B. Haughton, C. E., on "The paying and non-paying weights drawn by the locomotive engine in 1867."

Passenger Trains.

Paying weight, 4.89 per cent. of the total weight of the train.

Goods Trains.

Paying weight, 30.34 per cent. of the total weight of the train.

Total Passenger and Goods Trains.

Paying weight, 16:67 per cent. of the whole weight of trains.

Or, in other words, it takes 19 tons of train equipment to carry one ton of passengers, $2\frac{3}{4}$ tons of the same to carry one ton of goods, and in gross 5 tons of equipment to carry one ton of paying load.

"Thus," observes Mr. Haughton, "it appears from the returns that the average British passenger weighs two tons, with train accessories, and that the ton of goods weighs $3\frac{3}{4}$ tons; and by no known processes can this enormous multiplication of original net weights be reduced, consistent with affording that amount of personal security, comfort, and accommodation now enjoyed."

Upon the roads of New York State, in 1868, the average number of passengers per train is given as 70.81, and the average weight in tons of passenger trains, exclusive of passengers and baggage, 86.89 tons. The average number of tons in a freight train was 105.82; and the average weight in tons of freight trains, exclusive of freight, 146.72 tons.* From this it would seem that to carry one ton of passengers in New York State (reckoning 12 passengers with their baggage to the ton), it requires 14 tons of passenger train equipment, and to carry a ton of freight it requires 1.4 tons of freight train equipment; or the average passenger in New York State weighs 1.3 tons, and the average ton of freight 2.4 tons, very nearly. From the annual returns made by the Railroad Corporations of Massachusetts to the State, for 1869, the figures below are obtained, the dead weight of freight and passenger cars being, however, only estimated:—

* State Engineer's Report for 1868.

NAME OF ROAD	Passengers carried one Mile.	Tons of Pass'r Cars hauled one Mile.	Tons of Freight hauled one Mile.	Tons of Freight Cars hauled one Mile
Boston and Albany, .	95.678.232	51,028,384	158.579.177	190.800.000
Boston and Lowell, . .	18,587.217	9 293.608	12.330.813	18.496.219
Boston and Maine, . . .	51.248 678	18.265.297	16.163.410	20.276.543
Boston and Providence, .	29.963.489	14.981.745	13.021.748	19.532.622
Eastern Railroad, ...	57.357.609	11.471.522	8.896.106	5.337.664
Fitchburg Railroad, . . .	22.134.414	21.096.766	16.941.140	26.778.016
Old Colony and Newport,	41.484.089	15.508.795	10.810,178	17.217.999
Providence and Worcester,	12.258.435	5.177.260	12.300.450	16.719.170
Norwich and Worcester, .	7.312.851	6.929.100	11.065.863	19.307.715
Worcester and Nashua, .	5.287.044	3.115.200	7.835.998	13.329 562
	341,312.058	156.867.677	267.944.883	347.795.510

From these figures it appears that the transport of each passenger involves the moving of about half a ton, and each ton of freight the moving of 2.3 tons. We shall probably not be far from the truth in stating, generally, that in this country the transportation of each passenger involves the hauling of one ton of gross weight, and each ton of freight the hauling of 2½ tons. The reduction of this large proportion of non-paying weight is a most important problem for railway managers, but one which is not receiving the attention it deserves, especially in the passenger department, where the introduction of the new and excessively heavy drawing-room cars is at once augmenting the dead weight and reducing the paying load.*

* The heaviest drawing-room cars weigh no less than 35 tons without passengers. They accommodate a less number of persons than the ordinary car, but an extra charge is made for the increased accommodation.

FAST TRAINS.

Express trains, Dr. Lardner observed, in 1850, are a source of vast expense, directly and indirectly, which can never be repaid by any practicable tariff to be levied upon them. In 1854 a convention of American Railroad Managers recommended the adoption of a higher rate of fare upon express passenger trains, corresponding in some degree to the increased cost of such trains. The cost of running trains has been stated to increase nearly as the square of the speed. The wear and tear of engines, cars, and track certainly increases in a rapid ratio with the velocity, and in addition to this the influence of express trains is not confined to themselves, but all trains in motion at the same time within a certain distance of the express must be kept waiting with steam up, or be driven with extra velocities to keep out of the way. The running time may be reduced with much more economy by lessening the number of stops than by an actual higher velocity while running. A train to run 100 miles in four hours, making no stops, would run only 25 miles an hour; to make 10 stops of six minutes each, the speed would have to be $33\frac{1}{3}$ miles an hour; and to make 20 stops of six minutes each, and accomplish the 100 miles in four hours, would require a speed while running of 50 miles an hour. Mr. McCallum, in the report before referred to, makes the following remarks upon this subject:—

"In estimating the effect of high rates of speed in the cost of operating a road, it is not sufficient to count only the expense involved by the expenditure of the greater power required, and the additional wear and tear of the roadway and machinery, as these, though by no means unimportant items, may be considered as such when compared with the uncertain contingencies growing out of it, prominent among which are the delays caused by the increased liability of not reaching the stations by the time prescribed, and the accidents resulting from the effort to do so. Where the time table is so arranged as to call for speed nearly equal to the full capacity of the engine, it is very obvious that the

risks of failure in 'making time' must be much greater than at reduced rates; and when failures occur, the efforts made to gain time must be correspondingly greater and uncertain. A single example will be sufficient to show this.

"A train whose prescribed rate of speed is 30 miles an hour, having lost five minutes of time, and being required to gain it, in order to meet and pass an opposing train at a station 10 miles distant, must necessarily increase its speed to 40 miles an hour; and a train whose prescribed rate of speed is 40 miles an hour, under similar circumstances, must increase its speed to 60 miles an hour; in the former case it would probably be accomplished, whilst in the latter it would more probably result in failure; or, if successful, it would be so at a fearful risk of accident.

"But a failure in either case would have the effect of retarding the movement of the opposing train, deranging the time of those of the same and of an inferior class, in both directions, involving, perhaps, on the part of the latter the necessity of similar struggles for time, and thus may prove the primary cause of accident to all trains whose movements may have been affected thereby.

"In considering the additional expense above stated as amongst the incidents of high speed, we must not lose sight of another prominent effect of the irregularity and delay thus produced. If from the above cause freight trains are compelled to lay on switches for one fourth of their time, which is not unfrequently the case, the value of the fuel consumed, and the time of the men employed, is entirely lost to the company, and the rolling stock and facilities sufficient for the transaction of a given amount of business perform only three fourths of what they are susceptible under other circumstances. Reasonable speed and regularity are much more desirable than high speed and its attendants, irregularity and delay, which are frequently produced by the introduction of one fast train only.

"The above remarks have been offered in considering the effect

of high rates of speed on passenger trains, but are equally true as applied to those of freight. In fact, an equal increase of speed in freight trains, in consequence of the much greater weight of its engines, is more serious in its effect on the roadway and machinery than in the case of passenger trains."

Methods of Increasing Railway Profits.

To increase the economy of railway operation, the following points are to be regarded. *First*, to manage the traffic so as to cause the cars to carry more complete loads, thus to reduce the ratio of the non-paying weight. *Second*, to arrange the trains so as to give the largest number of cars to each engine. *Third*, the reduction of speed of both passenger and freight trains to the lowest standard consistent with the demands of the business. *Fourth*, to diminish as far as possible express trains, except so far as reducing the number of stops shortens the time of traversing the road. *Fifth*, not to increase the number of trains beyond a reasonable accommodation of traffic. *Sixth*, so controlling the movement of the rolling stock that the greatest amount of service may be derived from it. *Seventh*, adjusting the tariffs, where the business is chiefly in one direction, so as to attract return traffic, that the cars may not run without a load. *Eighth*, reducing the friction, by which the cost of repairs of rolling stock is diminished, and the facility for economical transportation proportionately increased.

The Telegraph.

Nothing has done more to enable railway superintendents to exercise complete control of the works in their charge than the telegraph. Indeed, no railway at the present day is complete, or even safe, without this important aid. Mr. McCallum thus refers to this subject:—

" A single track railroad may be rendered more safe and effi-

cient, by a proper use of the telegraph, than a double track railroad without its aid, as the double track can only obviate collisions which occur between trains moving in opposite directions, whilst the telegraph may be used effectually in preventing them, either from trains moving in an opposite or the same direction; and it is a well established fact, deduced from the history of railroads, both in Europe and this country, that collisions between trains moving in the same direction have proved by far the most fatal and disastrous, and should be the most carefully guarded against. I have no hesitation in asserting that a single track railroad, having judiciously located turnouts, equal in the aggregate to one quarter of its entire length, and a well-conducted telegraph, will prove to be a more safe and profitable investment than a much larger sum expended in the construction of a continuous double track operated without a telegraph.

"Collisions between fast and slow trains moving in the same direction are prevented by the application of the following rule: 'The conductor of a slow train will report himself to the superintendent of the division immediately on arrival at a station where by the time table he should be overtaken by a faster train; and he shall not leave that station until the fast train passes, without special orders from the superintendent of the division.' A slow train, under such circumstances, may, at the discretion of the division superintendent, be directed to proceed. He being fully apprised of the position of the delayed train, can readily form an opinion as to the propriety of doing so, and thus, whilst the delayed train is permitted to run without regard to the slow train, the latter can be kept entirely out of its way.

"In moving trains by telegraph nothing is left to chance. Orders are communicated to the conductors and engineers of the opposing trains, and their answers returned, giving their understanding of the order before either is allowed to proceed. Their passing place is fixed and determined with orders positive and defined, that neither shall proceed beyond that point until after the arrival of the other; whereas, in the absence of a telegraph,

conductors are governed by general rules, and their individual understanding of the same ; which rules are generally to the effect that in cases of detention the train arriving first at the regular passing place shall, after waiting a few minutes, proceed *cautiously*, 'expecting to meet the other train,' until they have met and passed, the one failing to reach the half-way post between stations being required to back (always a dangerous expedient), and the other permitted to proceed ; the delayed train being subjected to the same rule in regard to all other trains of the same class it may meet, thus pursuing its hazardous and uncertain progress during the entire trip. The history of such a system furnishes a serious commentary on the imperfection of railroad regulations.

"The liability to collision under the system referred to has prompted the invention of various expedients for suddenly arresting the progress of trains, which seem to have been conceived under the impression, more imaginary than real, that the difficulties they were designed to obviate are unavoidable in their character ; but which may, by the exercise of ordinary care, and the use of the telegraph, be subjected to perfect control. Some of these inventions undoubtedly possess sufficient merit to entitle them to adoption under any circumstances, whilst others, for the above reasons, are entirely valueless. Indeed, it is questionable whether a reliance on their use may not, in many cases, lead to danger, by producing recklessness, and thus increase instead of diminish the evils sought to be avoided."

TIME TABLES.

In preparing time tables, for the running of trains, a diagram or chart, like that shown in Fig. 165, is employed. In this chart *time* is denoted by movement from left to right, horizontally. Distances along the line of the road are denoted by movement up or down on the vertical lines. Any diagonal line upon the chart, therefore, represents movement, both in distance along the road, and also in time. In the figure, the heavy vertical lines repre-

sent the successive hours of the day, and the intermediate finer lines the quarter hours. The horizontal lines represent the several stations along the road, the vertical distances between these being plotted by scale, according to the actual distances in miles. A common scale in practice for plotting such a diagram is one inch per hour, horizontally, and eight miles to the inch, vertically.

Suppose that we wish to start a train at 6 A. M., from the station represented by A A, so that it shall arrive at station J at 3 P. M., stopping 15 minutes at each way station. The number

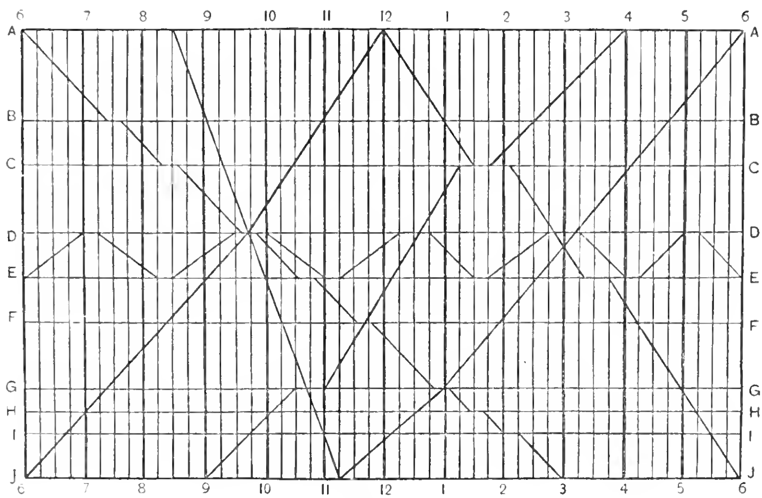

Fig. 165.

of way stations being 8, the whole time consumed by stops, will be 120 minutes, or 2 hours. From 3 P. M., on the lower horizontal line, go back 2 hours, or to 1 P. M ; and from 6 A. M., on the upper line, draw a line, which produced would hit 1 P. M. on the lower line. This diagonal reaches the line B B at 7.25. As we stop 15 minutes at the station, we pass along *on the line* B B a distance equal to 15 minutes on the time scale, and from the point thus reached we start again parallel to the first diagonal, arriving at station C at 8.20. Proceeding in the same way, we

arrive at station J at 3 P. M., as desired. The rate of inclination of the diagonal shows the speed.

If we would start a train from A at 8.30 to arrive at J at 11.15, making no stops, it will pass the train above mentioned at station D, and will run the whole distance in 2 hours 45 minutes. Trains running in the opposite direction are represented on the diagram by diagonals ascending from left to right. Thus, a train leaving station J at 6 A. M., to arrive at A at noon, making no stops, will run, as by the broken diagonal, from 6 A. M., on the lower line, to 12 on the upper one, passing the 6 A. M. and the 8.30 A. M. trains, running in the opposite direction, at station D. It will be observed that the line from 6 to 12 changes its rate of inclination at the horizontal D; by which we understand that the train changes its rate of speed at that station, running faster from D to A than from J to D.

If it is desired to work a construction train between stations E and D, from 6 A. M. to 6 P. M., the movement of such a train is shown by the short diagonals between the horizontals D and E, and its time card would be thus: Leave E at 6 A. M., and arrive at D at 7. Leave D at 7.15, and arrive at E at 8.15. Leave E at 8.30, and arrive at D at 9.30; crossing the 6 and the 8.30 A. M. trains from A to J, and being passed by the 6 A. M. train from J to A. Leave D at 10, and arrive at E at 11. Leave E at 11.15, and arrive at D at 12.15, and wait to be passed by 9 A. M. train, from J to A. Leave D at 12.45 P. M., and arrive at E at 1.30. Leave E at 1.45, arrive at D at 2.45, and pass noon train from station A, and 11.15 A. M. train from station J. Leave D at 3.15, and arrive at E at 4 P. M. Leave E at 4.15 P. M., and arrive at D at 5. Leave D at 5.15, and arrive at E at 6 P. M.

If a train leaves A at noon, and runs towards J, leaving C at 2.05, and reaching E at 3.20, and another train leaves J at 11.15 A. M. and G at 1 P. M., and runs to A, as by the diagonal, without stopping, the trains will pass at 3 P. M. at a point between D and E, the exact position of which may be found by the scale of miles, according to which the length of the road, or the dis-

tance A J, is plotted; at which place a siding or passing place must be provided.

Upon a double track road a chart may be prepared for each track, and diagonals in one direction only will appear upon each diagram.

In practice, the diagram is accurately drawn to a large scale, and the several trains are represented by differently colored elastic lines fastened by pins, so that they may be moved from hour to hour through the day and night as the various occurrences on the road may demand; some trains being retarded, others hastened, extras put in, and all provisions made for securing regularity in the movement, and freedom from disaster.

The grades and curves may, if desirable, be shown upon the vertical line A J; by which those parts of the road may at once be seen where, from increased resistance, a lower speed will need to be adopted.

APPENDIX.

I.

EXPLANATION OF THE SIGNS USED IN ALGEBRAIC FORMULÆ.

$=$ denotes equality; as, 4 times $6 = 24$.

$+$ denotes addition; as, $4 + 6 = 10$.

$-$ denotes subtraction; as, $10 - 2 = 8$.

$\times$ denotes multiplication; as, $10 \times 4 = 40$.

$\div$ denotes division; as, $24 \div 6 = 4$; and $\frac{24}{6}$ denotes the same.

: :: : signifies ratio or proportion; as, 4 : 12 :: 6 : 18, or 4 is to 12 as 6 is to 18.

8^2 denotes that 8 is to be multiplied by itself once; or, $8 \times 8 = 64$.

8^3 shows that 8 is to be multiplied by itself twice; or, $8 \times 8 \times 8 = 512$.

$\sqrt{16}$ denotes the square root of 16, or 4.

$\sqrt[3]{64}$ denotes the cube root of 64, or 4.

$\frac{a + b + c}{d}$, shows that the sum of a, b, and c is to be divided by d.

$(a + b + c)\,d$ or $\overline{a + b + c} \times d$, denotes that the sum of a, b, and c is to be multiplied by d.

Generally, in place of writing $a \times b$ to express multiplication, we put simply $a\,b$.

In Chapter VIII. we have the following formula:—

$$P = \frac{f\,S}{1 + a\frac{l^2}{h^2}},$$

denoting that the value of l is to be squared and divided by the value of h squared, and the quotient multiplied by a and added to 1; and, finally, the

result thus obtained is to be put into the product of f multiplied by S, the quotient being the value of P.

The above signs may also be compounded as in the formula below: —

$$\sqrt[4]{\frac{\left[\frac{(a+b)c}{d}\right]\frac{3}{4}}{m}}$$

Here we have, first, the product of c by the sum of a and b; this is divided by d, and three quarters of the quotient is divided by m; and, finally, the fourth root of the last result is extracted, which is the value of the expression.

II.

WEIGHT OF MATERIALS.

WEIGHT of a cubic foot of various substances in pounds avoirdupois.

Air, under a pressure of 1 atmosphere, ...	.0807
Brass, cast, ...	486 to 525
Brick, ...	125 to 135
Brickwork, ...	112
Bronze, ...	524
Chalk, ...	117 to 174
Clay, ...	120
Coal, anthracite, ...	100
Coal, bituminous, ...	78 to 90
Coke, ...	63 to 104
Copper, cast, ...	537
Gold, ...	1186 to 1224
Granite, ...	164 to 172
Iron, cast, ...	434 to 456
Iron, wrought, ...	474 to 487
Lead, ...	712
Limestone, ...	170 to 180
Masonry, ...	116 to 144
Mercury, ...	848
Mortar, ...	109
Mud, ...	102
Platinum, ...	1311 to 1373
Sand, damp, ...	118
Sand, dry, ...	88

Sandstone,	130 to 157
Silver,	655
Slate,	175 to 181
Steel,	487 to 493
Tin,	456 to 468
Trap-rock,	170
Wood, hard and dry,	45 to 65
Wood, hard and green,	60 to 80
Wood, soft and dry,	30 to 45
Wood, soft and green,	40 to 60
Zinc,	424 to 449

III.

TABLE OF GRADES.

Grade in Feet per Mile.	Rise per 100 Feet.	Grade in Feet per Mile.	Rise per 100 Feet.	Grade in Feet per Mile.	Rise per 100 Feet.	Grade in Feet per Mile.	Rise per 100 Feet.
2.64	.05	55.44	1.05	110.88	2.10	216.48	4.10
5.28	.10	58.08	1.10	116.16	2.20	221.76	4.20
7.92	.15	60.72	1.15	121.44	2.30	227.04	4.30
10.56	.20	63.36	1.20	126.72	2.40	232.32	4.40
13.20	.25	66.00	1.25	132.00	2.50	237.60	4.50
15.84	.30	68.64	1.30	137.28	2.60	242.88	4.60
18.48	.35	71.28	1.35	142.56	2.70	248.16	4.70
21.12	.40	73.92	1.40	147.84	2.80	253.44	4.80
23.76	.45	76.56	1.45	153.12	2.90	258.72	4.90
26.40	.50	79.20	1.50	158.40	3.00	264.00	5.00
29.04	.55	81.84	1.55	163.68	3.10	269.28	5.10
31.68	.60	84.48	1.60	168.96	3.20	274.56	5.20
34.32	.65	87.12	1.65	174.24	3.30	279.84	5.30
36.96	.70	89.76	1.70	179.52	3.40	285.12	5.40
39.60	.75	92.40	1.75	184.80	3.50	290.40	5.50
42.24	.80	95.04	1.80	190.08	3.60	295.68	5.60
44.88	.85	97.68	1.85	195.36	3.70	300.96	5.70
47.52	.90	100.32	1.90	200.64	3.80	306.24	5.80
50.16	.95	102.96	1.95	205.92	3.90	311.52	5.90
52.80	1.00	105.60	2.00	211.20	4.00	316.80	6.00

TABLE FOR BENDING RAILS FOR CURVES.

Degree of Curvature.	Radius in Feet.	Versed Sine, or Middle Ordinate in Inches, the length of Rail in Feet being — 12	14	16	18	20	22	24
1	5730	1/32	1/16	1/16	1/16	1/8	1/8	1/8
2	2865	1/16	1/8	1/8	3/16	3/16	1/4	5/16
3	1910	1/8	1/8	3/16	1/4	5/16	3/8	7/16
4	1433	1/8	3/16	1/4	5/16	7/16	1/2	5/8
5	1146	3/16	1/4	5/16	7/16	1/2	5/8	3/4
6	955	1/4	5/16	3/8	1/2	5/8	3/4	7/8
7	819	1/4	3/8	1/2	9/16	3/4	7/8	1 1/16
8	717	5/16	7/16	9/16	11/16	13/16	1	1 3/16
9	637	5/16	7/16	5/8	3/4	15/16	1 1/8	1 3/8
10	574	3/8	1/2	11/16	7/8	1 1/16	1 1/4	1 1/2
11	522	7/16	9/16	3/4	15/16	1 1/8	1 3/8	1 5/8
12	478	7/16	5/8	13/16	1	1 1/4	1 1/2	1 13/16
13	442	1/2	11/16	7/8	1 1/8	1 3/8	1 5/8	1 15/16
14	410	1/2	11/16	15/16	1 3/16	1 7/16	1 3/4	2 1/8
15	383	9/16	3/4	1	1 1/4	1 9/16	1 7/8	2 1/4
16	359	5/8	13/16	1 1/16	1 3/8	1 11/16	2	2 7/16
17	338	5/8	7/8	1 1/8	1 7/16	1 3/4	2 1/8	2 9/16
18	320	11/16	15/16	1 3/16	1 1/2	1 7/8	2 1/4	2 11/16
19	303	11/16	1	1 1/4	1 5/8	2	2 3/8	2 7/8
20	288	3/4	1	1 5/16	1 11/16	2 1/16	2 1/2	3

TABLE OF ORDINATES UPON A HUNDRED FEET CHORD, FOR SHARP CURVES.

Degree of Curvature.	Radius in Feet.	Ordinates in Decimals of a Foot at a Distance of			
		12½ Feet.	25 Feet.	37½ Feet.	50 Feet
1	5730	.095	.164	.205	.218
2	2865	.191	.327	.409	.436
3	1910	.286	.491	.614	.655
4	1433	.382	.655	.818	.873
5	1146	.477	.818	1.023	1.091
6	.955	.573	.982	1.228	1.309
7	.819	.669	1.146	1.432	1.528
8	.717	764	1.310	1.637	1.746
9	.637	.860	1.474	1.842	1.965
10	.574	.956	1.638	2 047	2.183
11	.522	1.052	1.802	2.252	2.402
12	.478	1.148	1.967	2.457	2.620
13	.442	1.244	2.131	2.662	2.839
14	.410	1.341	2.296	2.868	3 058
15	.383	1.437	2.461	3.073	3.277
16	.359	1.534	2.625	3.279	3.496
17	.338	1.631	2.791	3.485	3.716
18	.320	1.728	2.956	3 691	3.935
19	.303	1.825	3.121	3.897	4.155
20	.288	1.922	3.287	4.103	4.374

TABLE OF ANGLES AND DISTANCES FOR FROGS.

Gauge, 4 feet $8\frac{1}{2}$ inches. Length of switch rail, 18 feet. Movement, 5 inches.

Radius of Curve.	Angle of Frog	Length of Chord.	Radius of Curve.	Angle of Frog.	Length of Chord.
1000	5° 28′	72.2	600	6° 58′	59.2
950	5° 36′	70.8	550	7° 16′	57.1
900	5° 44′	69.4	500	7° 36′	54.9
850	5° 54′	67.9	450	8° 00′	52.6
800	6° 04′	66.3	400	8° 28′	50.1
750	6° 15′	64.6	350	9° 02	47.3
700	6° 28′	62.9	300	9° 45′	44.3
650	6° 42′	61.1	250	10° 39′	41.0

The length of chord given above, is the distance from the point of the frog to the movable end of the outer switch rail.

IV.

VALUES OF THE BIRMINGHAM AND AMERICAN GAUGES

Employed for designating the Thickness of Iron Wire, Sheet Iron, and Steel.

No.	Thickness in Inches. English.	American.	No.	Thickness in Inches. English.	American.	No.	Thickness in Inches. English.	American.
1	.300	.289	13	.095	.072	25	.020	.018
2	.284	.258	14	.083	.064	26	.018	.016
3	.259	.229	15	.072	.057	27	.016	.014
4	.238	.204	16	.065	.051	28	.014	.013
5	.220	.182	17	.058	.045	29	013	.011
6	.203	.162	18	.049	.040	30	.012	.010
7	.180	.144	19	.042	.036	31	.010	.009
8	.165	.128	20	.035	.032	32	.009	.008
9	.148	.114	21	.032	.028	33	.008	.007
10	.134	.102	22	.028	.025	34	.007	.006
11	.120	.091	23	.025	.023	35	.005	.006
12	.109	.081	24	.022	.020	36	.004	.005

Besides the above, No. 0 is .340; 00, .380; 000, .425; and 0000, .454 of an inch.

TABLE OF WHITWORTH'S SCREWS.

[Dimensions are all in inches.]

Diameter of Bolt.	Threads per Inch.	Diameter of Nut.	Thickness of Nut.	Diameter of Head.	Thickness of Head.
$\frac{1}{4}$	20	$\frac{3}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{4}$
$\frac{5}{16}$	18	$\frac{9}{16}$	$\frac{5}{16}$	$\frac{9}{16}$	$\frac{5}{16}$
$\frac{3}{8}$	16	$\frac{11}{16}$	$\frac{3}{8}$	$\frac{9}{16}$	$\frac{5}{16}$
$\frac{1}{2}$	12	$\frac{7}{8}$	$\frac{1}{2}$	$\frac{3}{4}$	$\frac{7}{16}$
$\frac{5}{8}$	11	$1\frac{1}{16}$	$\frac{5}{8}$	$\frac{15}{16}$	$\frac{9}{16}$
$\frac{3}{4}$	10	$1\frac{1}{4}$	$\frac{3}{4}$	$1\frac{1}{8}$	$\frac{5}{8}$
$\frac{7}{8}$	9	$1\frac{7}{16}$	$\frac{7}{8}$	$1\frac{5}{16}$	$\frac{3}{4}$
1	8	$1\frac{5}{8}$	1	$1\frac{1}{2}$	$\frac{7}{8}$
$1\frac{1}{8}$	7	$1\frac{13}{16}$	$1\frac{1}{8}$	$1\frac{11}{16}$	1
$1\frac{1}{4}$	7	2	$1\frac{1}{4}$	$1\frac{7}{8}$	$1\frac{1}{8}$
$1\frac{3}{8}$	6	$2\frac{3}{16}$	$1\frac{3}{8}$	$2\frac{1}{16}$	$1\frac{1}{4}$
$1\frac{1}{2}$	6	$2\frac{3}{8}$	$1\frac{1}{2}$	$2\frac{1}{4}$	$1\frac{3}{8}$
$1\frac{5}{8}$	5	$2\frac{9}{16}$	$1\frac{5}{8}$	$2\frac{7}{16}$	$1\frac{1}{2}$
$1\frac{3}{4}$	5	$2\frac{3}{4}$	$1\frac{3}{4}$	$2\frac{5}{8}$	$1\frac{5}{8}$
$1\frac{7}{8}$	$4\frac{1}{2}$	$2\frac{15}{16}$	$1\frac{7}{8}$	$2\frac{13}{16}$	$1\frac{3}{4}$
2	$4\frac{1}{2}$	$3\frac{1}{8}$	2	3	$1\frac{7}{8}$
$2\frac{1}{4}$	$4\frac{1}{2}$	$3\frac{1}{2}$	$2\frac{1}{4}$	$3\frac{3}{8}$	2
$2\frac{1}{2}$	4	$3\frac{7}{8}$	$2\frac{1}{2}$	$3\frac{3}{4}$	$2\frac{1}{4}$
$2\frac{3}{4}$	4	$4\frac{1}{4}$	$2\frac{3}{4}$	$4\frac{1}{8}$	$2\frac{1}{2}$
3	$3\frac{1}{2}$	$4\frac{5}{8}$	3	$4\frac{1}{2}$	$2\frac{3}{4}$

The American standard screws are given in Chapter IX. of this work.

V.

PHŒNIX COLUMNS.

No. of Segments in Columns.	Diameter on Neutral Line.	Thickness of Metal.	Area in Square Inches.	Weight in Pounds per Yard.
4	$3\frac{13}{16}$	$\frac{1}{8}$	2.75	27.6
4	$3\frac{13}{16}$	$\frac{5}{16}$	5.50	54.9
4	$5\frac{1}{8}$	$\frac{3}{16}$	5.75	57.6
4	$5\frac{1}{8}$	$\frac{1}{2}$	12.12	121.2
4	$7\frac{9}{16}$	$\frac{1}{4}$	11.25	112.5
4	$7\frac{9}{16}$	$\frac{7}{8}$	28.88	288.9
5	$9\frac{9}{16}$	$\frac{5}{16}$	15.94	159.3
5	$9\frac{9}{16}$	$\frac{15}{16}$	37.65	376.5
6	$11\frac{1}{2}$	$\frac{5}{16}$	19.78	198.0
6	$11\frac{1}{2}$	1	47.81	477.6
7	$13\frac{3}{8}$	$\frac{3}{8}$	26.47	264.0
7	$13\frac{3}{8}$	$1\frac{1}{16}$	58.41	584.4
8	$15\frac{1}{4}$	$\frac{3}{8}$	30.25	302.1
8	$15\frac{1}{4}$	$1\frac{1}{16}$	65.79	657.9

A new wrought iron column, designed by Mr. Charles Bender, C. E., of Phœnixville, consists of a number of segments with their edges rolled down so as to make a flange projecting inside of the column. A clamp or channel iron embraces these internal flanges, running the whole length of the column, and is thus available for resisting compression. These members are connected by bolts or dowels, with countersunk heads outside, passing through the joint between the flanges, and secured by nuts on the inside of the channel iron. The bearing between the clamp or channel bar and the flanges is so bevelled that screwing the nut draws the flanges close. This column is simple in its

construction, is entirely smooth on the outside, affords no chance for the lodgment of water where it is employed as a horizontal member, and by cement or calking may be made even air or water tight, and thus secure against rust inside. The method of making these columns has received particular attention, and the question of cost in all its details has shown that the columns can be offered at a reasonable price.

VI.

ON THE SO-CALLED CRYSTALLIZATION OF WROUGHT IRON.

UPON no subject has more difference of opinion been expressed among engineers than upon the so-called crystallization or granulation of wrought iron when subjected to long-continued vibrations or concussions. The breakage of railway axles has called frequent and especial attention to this matter, as they are subjected to severe and rapid shocks, and, when they break, invariably present the so-termed crystalline appearance. This change, from a tough and fibrous bar to a brittle, granular, or crystalline one, has been attributed to concussion, vibration, sudden cooling, heat, or magnetism, simply or in combination. Much confusion has arisen from the different appearances of fracture shown by different qualities of iron, and from the different modes of describing those appearances. Axles have broken off as short as glass at one end, while the other parts were tough and fibrous. Mr. J. E. McConnell (England), who has had great experience upon this point, considers the fact well established that some change in the nature of the iron does take place, in which he is supported by a large amount of evidence. Mr. Bourville made experiments under direction of the Austrian government, in which seven axles were subjected to repeated torsions and shocks, with the following results: The first axle received 32,400 shocks, and then, being broken in a hydraulic press, no change in the structure of the iron was visible. The second axle received 129,000 torsions, and when broken, though no change could be seen by the naked eye, under the microscope "the fibres appeared without adhesion, like a bundle of needles." The third axle received 338,000 torsions, and when broken, "a change in its texture and an increased size in the grain of the iron was observable by the naked eye." The fourth axle received 2,588,000 torsions, and being broken, "a considerable change in the texture was apparent, which was more striking towards the centre, and the size of the grains diminished towards the extremities." The fifth axle received 23,328,000 torsions, and "was completely changed in its texture." The sixth axle received 78,732,000 torsions and shocks, and the subsequent fracture showed clearly "an absolute transformation of the structure of the iron, the surface of the rupture being scaly like pewter." The seventh axle received 128,304,000 torsions, and upon the

surface of rupture "the crystals were found to be perfectly well defined, the iron having lost every appearance of wrought iron."

Mr. Robert Stephenson was unable to satisfy himself, from a large experience, that any such molecular change in fibrous iron took place, and referred to the beam of a Cornish engine, which, working eight or ten strokes a minute for more than twenty years, under a strain of fifty pounds per inch, and the connecting rod of a locomotive, vibrating eight times a second for several years of regular work, making more than 200,000,000 times, yet remained uninjured; and he considers these facts good grounds for doubting that iron changes its state in axles. Many engineers have doubted that any axle which, when broken, proved to be crystalline, had ever been fibrous in its character. Others maintain that the change does not take place unless the iron is strained beyond its limit of elasticity. One of the most striking examples of the change is shown in the chain slings used for carrying the bars during the process of hammering at a forge, and also in the porter bars attached to the blooms while under the hammer, both of which are known to become very brittle after a few months. Mr. Thorneycroft (England) considered that the internal structure of iron undergoes no change unless there be a change of form; that simple vibration will not destroy the fibre, whereas bending, if long continued, would change the most fibrous iron to crystalline. Mr. Roebling states that the most fibrous bar may be broken so as to show a granular and somewhat crystalline fracture, and this without undergoing any molecular change in the texture. "Take," he says, "a fibrous bar, say ten feet long, nip it in the centre all around with a cold chisel, then poise the bar upon the short edge of a large anvil and a short piece of iron placed eight or nine inches from the edge on the face of the anvil, and strike a few heavy blows upon the nip, so that each blow will cause the bar to rebound and to vibrate intensely, and the result will be a granular and somewhat crystalline fracture. Now take up the two halves and nip them all round again, about one or two inches from the fractured ends, and break them off by easy blows over the *round* edge of the anvil, and the fibre will appear again. This experiment proves that a break caused by sudden jars and intense vibration may show a granular and even crystalline fracture, without having changed the molecular arrangement of the iron. All fibres are composed of mineral crystals drawn out and elongated or flattened, and the fracture may be produced so as to exhibit in the same bar, and within the same inch of bar, either more fibre or more crystal. But a coarse crystalline bar will under no circumstances exhibit fibre, nor will a well worked out fibre exhibit coarse crystals." Mr. Roebling concludes that a molecular change, or so-called granulation or crystallization, in consequence of vibration, or tension, or both combined, has in no instance been satisfactorily proved or demonstrated by experiments; but that vibration and tension combined will greatly affect the strength of iron *without* changing its fibrous texture, and

that this loss of strength bears a due proportion to the extent and duration of the vibration and tension.

Mr Kirkaldy observes that, "First, whenever wrought iron breaks *suddenly*, a crystalline appearance is the invariable result; when *gradually*, invariably a fibrous appearance. Second, whether on the one hand it is finely or coarsely crystalline, or on the other the fibre be fine and close, or coarse and open, depends upon the quality of the iron. Third, when there is a combination in the same bar or plate of two kinds, the one harder or less ductile than the other, the appearance will be partly crystalline and partly fibrous, the latter produced by the gradual drawing asunder action previous to and at the time of rupture, whilst in the former the iron breaks suddenly without elongating at the time of rupture. Fourth, when the proportion of the harder is considerably less than the softer, the former snaps suddenly whilst the latter continues stretching; but when nearly equal, or the less ductile predominates, both portions break together, or almost at the same moment; the one part gradually arriving at the limit of endurance, breaks with a fibrous appearance, whilst a greatly increased strain consequently coming on the remaining portion, it suddenly gives way, producing a crystalline appearance. Fifth, the relative qualities of various irons may be pretty accurately judged of by comparing their fractures, provided they have all been treated in precisely the same way, and all broken under the same sort of strains similarly applied. Sixth, by varying either the shape, the treatment, the kind of strain or its application, pieces cut off the same bar will be made to present vastly different appearances in some kinds of iron, whilst in others little or no difference will result."

According to Mr. Kirkaldy, it is not to any of the various forces suggested that the so-called crystalline appearance is due, but to *the act of breaking*. That some of the causes should have operated badly on the iron, and so facilitated rupture, he regards as a totally different question, which has been somewhat unaccountably mixed up with the former. Mr. Stephenson could not "conceive of a change going on in the structure of iron, as it would involve a change from one kind of crystalline structure to another, which was next to an impossibility." Mr. Roebling observes that "crystallization in iron, or any other metal, *can never take place in a cold state*." It is, however, well known at this day that the most extreme molecular changes *do* take place in solid bodies. "The maximum strength and toughness of iron," says Mr. Roebling, "is obtained by a certain mixture of pure iron with carbon and cinder, thoroughly worked. The drawn-out fibre is composed of an aggregate of pure iron threads and leaves enveloped in cinder. Even the best fibrous wrought iron, when exposed to long continued vibration under tension or twisting, must inevitably become brittle, *because the iron threads and laminæ become loosened in their cinder envelopes*. The cohesion between the iron and its cinder once destroyed, its strength is gone." This process destroying the cohesion Mr. Roebling considers to be purely mechanical, and he concludes.

"Let the explanation here given be correct or not, the fact remains that fibrous iron, and all kinds of iron and steel, will be rendered brittle by vibration and tension, or by bending and twisting, *without* undergoing any mysterious change in its molecular arrangement."

The presence of cinder can hardly have the importance attached to it by Mr. Roebling, as the homogeneous Bessemer iron, which is stronger than the ordinary fibrous iron, has very little if any cinder. Indeed, Mr. Roebling himself states that soft steel, with more of a granular texture than fibre, possesses a much greater elasticity and strength than the best fibrous iron.

From the preceding memoranda we may conclude that the best fibrous iron will, by repeated and long continued vibration, under severe strains, be so affected as to break suddenly; and that when it does so, it will show the so-called "crystalline" structure. Vibration, or bending and twisting, will, according to Mr. Roebling, render all kinds of iron and steel brittle, "without undergoing any mysterious change in molecular arrangement," and even if the "crystallization," as Mr. Kirkaldy states it, is due to the *act of breaking*, being present with a sudden fracture, and not with a slow one, inasmuch as the act of breaking is itself the result of the brittleness produced by the vibration, we must conclude that the repeated shocks, the becoming brittle, the sudden fracture, and the crystalline appearance, are successive steps in the same operation.

With regard to the effect of vibration in bridge work, rods have repeatedly been tested after from twenty to thirty years of service, and found as good as new; so that there is no reason to suppose that the iron in bridges will fail from the vibrations to which they are subjected in practice, if a sufficient margin of safety is allowed. With regard to axles, the important point seems to be to provide *such a form* that the vibrations shall be spread over the whole axle, and not be allowed to concentrate at particular places.

VII.

TESTING IRON BY MAGNETISM.

A METHOD of detecting faults in iron forgings by means of an examination of the bar, shaft, or whatever the work may be, with a magnetic needle, has been employed for some time in England, and not only does this method detect a fault in the weldings, but it indicates the change which so often occurs in iron from the fibrous to the so-called crystalline condition. The great value of this discovery will be fully appreciated by all persons who have dealings with shafting, axles, and the various combinations of machinery which depend entirely for safety upon the integrity of the iron work used

in the construction. The method was discovered by Mr. Saxby, R. N., and has been tested upon a great variety of forgings at the royal dockyards at Sheerness and Chatham. The process depends upon the very simple principle, that when a bar or mass of soft, homogeneous iron, of the best quality, and free from any flaws or defects, causing a separation of the particles of the iron, is placed in the position of the dipping-needle, it is at once sensibly magnetic — the lower end being a north pole in northern latitudes, and the upper end a south pole. The same action takes place in a bar hanging vertically, or in any other position, but to a less extent. With internal flaws, however, the bar is no longer one regular magnet, but several different magnets, with the different magnetisms separated from each other. When a delicately balanced magnetic needle is passed over the bar to be examined, the bar being placed east and west in the equatorial magnetic plane, if the bar is entirely sound the needle remains at right angles with the bar, that is, N. and S.; but the moment a flaw, or a separation between the particles of the iron, exists, the needle departs from its normal position and assumes a new direction, thus showing the place of the fault. A large number of trials at the shops of the royal dockyards were made in the presence of many engineers and iron-workers, and the place of the fault indicated by a chalk mark. Afterwards the bars were broken in a testing machine, and the result in every case showed the decision of the magnet to be entirely correct. In one case a round bar fourteen inches long had a hole drilled into it, and a bolt of unmagnetized steel was inserted and welded up with the end of the iron bar. The needle detected the fact that a fault existed at the place where the bolt was put in; and upon cutting up the bar it was found that one end of the bolt was not welded to the iron.

A large paddle crank-shaft was decided by the magnet to be defective near one neck, and, upon turning the metal down, the defect was found. A bar was decided by the test to be bad, and it was afterwards found to have been made partly of good and partly of bad iron. Another bar was declared faulty, and was found to be solid, but "upset" in the middle of its length, and then hammered down to its original diameter at a temperature *below* welding heat.

The experiments upon rolled plates, upon steel and cast iron, as far as made, are satisfactory, though further examination is needed to make the test practically available in these respects. Perhaps the most valuable result of this mode of testing iron is the detecting of that change called crystallization, which is so often seen in iron, and is so important a matter to be regarded in the use of railway axles.

VIII.

KIRKALDY'S CONCLUSIONS REGARDING THE QUALITIES OF IRON AND STEEL.

1. The breaking strain does *not* indicate the quality, as hitherto assumed.

2. A *high* breaking strain may be due to the iron being of superior quality, dense, fine, and moderately soft, or simply to its being very hard and unyielding.

3. A *low* breaking strain may be due to looseness and coarseness in the texture, or to extreme softness, although very close and fine in quality.

4. The contraction of area at fracture, previously overlooked, forms an essential element in estimating the quality of specimens.

5. The respective merits of various specimens can be correctly ascertained by comparing the breaking strain *jointly* with the contraction of area.

6. Inferior qualities show a much greater variation in the breaking strain than superior.

7. Greater differences exist between small and large bars in coarse than in fine varieties.

8. The prevailing opinion of a rough bar being stronger than a turned one is erroneous.

9. Rolled bars are slightly hardened by being forged down.

10. The breaking strain and contraction of area of iron plates are greater in the direction in which they are rolled than in a transverse direction.

11. A very slight difference exists between specimens from the centre and specimens from the outside of crank-shafts.

12. The breaking strain and contraction of area are greater in those specimens cut lengthwise out of crank-shafts than in those cut crosswise.

13. The breaking strain of steel, when taken alone, gives no clew to the real qualities of various kinds of that metal.

14. The contraction of area, at fracture, of specimens of steel must be ascertained, as well as in those of iron.

15. The breaking strain, *jointly* with the contraction of area, affords the means of comparing the peculiarities in various lots of specimens.

16. Some descriptions of steel are found to be very hard, and, consequently, suitable for some purposes; whilst others are extremely soft, and equally suitable for other uses.

17. The breaking strain and contraction of area of *puddled*-steel plates, as in iron plates, are greater in the direction in which they are rolled; whereas, in *cast*-steel they are less.

18. Iron, when fractured suddenly, presents invariably a crystalline appearance; when fractured slowly, its appearance is invariably fibrous.

19. The appearance may be changed from fibrous to crystalline by merely altering the shape of the specimen so as to render it more liable to snap.

20. The appearance may be changed by varying the treatment so as to render the iron harder and more liable to snap.

21. The appearance may be changed by applying the strain so suddenly as to render the specimen more liable to snap, from having less time to stretch.

22. Iron is less liable to snap the more it is worked and rolled.

23. The "skin," or outer part of the iron, is somewhat harder than the inner part, as shown by the appearance of fracture in rough and turned bars.

24. The mixed character of the scrap iron, used in large forgings, is proved by the singularly varied appearance of the fractures of specimens cut out of crank-shafts.

25. The texture of various kinds of wrought iron is beautifully developed by immersion in dilute hydrochloric acid, which, acting on the surrounding impurities, exposes the metallic portion alone for examination.

26. In the fibrous fractures the threads are drawn out and are viewed externally, whilst, in the crystalline fractures, the threads are snapped across in clusters, and are viewed internally or sectionally. In the latter cases the fracture of the specimen is always at right angles to the length; in the former it is more or less irregular.

27. Steel invariably presents, when fractured slowly, a silky, fibrous appearance; when fractured suddenly, the appearance is invariably granular, in which case, also, the fracture is always at right angles to the length; but when the fracture is fibrous the angle diverges, always more or less, from ninety degrees.

28. The granular appearance presented by steel suddenly fractured is nearly free of lustre, and unlike the brilliant crystalline appearance of iron suddenly fractured; the two combined in the same specimen are shown in iron bolts partly converted into steel.

29. Steel which previously broke with a silky, fibrous appearance, is changed into granular by being hardened.

30. The little additional time required in testing those specimens, whose rate of elongation was noted, had no injurious effect in lessening the amount of breaking strain, as imagined by some.

31. The rate of elongation varies, not only extremely in different qualities, but also to a considerable extent in specimens of the same brand.

32. The specimens were generally found to stretch equally throughout their length until close upon rupture, when they, more or less, suddenly drew out, usually at one part only, sometimes at two, and, in a few exceptional cases, at three different places.

33. The ratio of ultimate elongation may be greater in short than in long

bars, in some descriptions of iron, whilst in others the ratio is not affected by difference in the length.

34. The lateral dimensions of specimens form an important element in comparing either the rate of, or the ultimate, elongations — a circumstance which has been hitherto overlooked.

35. Steel is reduced in strength by being hardened in water, whilst the strength is vastly increased by being hardened in oil.

36. The higher steel is heated (without, of course, running the risk of being burned), the greater is the increase of strength by being plunged into oil.

37. In a highly converted or hard steel the increase in strength and in hardness is greater than in a less converted or soft steel.

38. Heated steel, by being plunged into oil instead of water, is not only considerably *hardened*, but *toughened* by the treatment.

39. Steel plates, hardened in oil and joined together with rivets, are fully equal in strength to an unjointed soft plate, or the loss of strength by riveting is more than counterbalanced by the increase in strength by hardening in oil.

40. Steel rivets, fully larger in diameter than those used in riveting iron plates of the same thickness, being found to be greatly too small for riveting steel plates, the probability is suggested that the proper proportion for iron rivets is not as generally assumed, a diameter equal to the thickness of the two plates to be joined.

41. The shearing strain of steel rivets is found to be about a fourth less than the tensile strain.

42. Iron bolts, case-hardened, bore a less breaking strain than when wholly iron, owing to the superior tenacity of the small proportion of steel being more than counterbalanced by the greater ductility of the remaining portion of iron.

43. Iron highly heated, and suddenly cooled in water, is hardened, and the breaking strain, when gradually applied, increased, but, at the same time, it is rendered more liable to snap.

44. Iron, like steel, is softened and the breaking strain reduced by being heated and allowed to cool slowly.

45. Iron, subjected to the cold-rolling process, has its breaking strain greatly increased by being made extremely hard, and not by being "consolidated," as previously supposed.

46. Specimens cut out of a crank-shaft are improved by additional hammering.

47. The galvanizing or tinning of iron plates produces no sensible effects on plates of the thickness experimented on. The results, however, may be different should the plates be extremely thin.

48. The breaking strain is materially affected by the shape of the specimen. Thus the amount borne was much less when the diameter was uniform for

some inches of the length, than when confined to a small portion — a peculiarity previously unascertained and not even suspected.

49. It is necessary to know correctly the exact conditions under which any tests are made before we can equitably compare results obtained from different quarters.

50. The startling discrepancy between experiments made at the Royal Arsenal and by the writer, is due to the difference in the shape of the respective specimens, and not to the difference in the two testing machines.

51. In screwed bolts the breaking strain is found to be greater when old dies are used in their formation than when the dies are new, owing to the iron becoming harder by the great pressure required in forming the screw thread when the dies are old and blunt than when new and sharp.

52. The strength of screw bolts is found to be in proportion to their relative areas, there being only a slight difference in favor of the smaller, compared with the larger sizes, instead of the very material difference previously imagined.

53. Screw bolts are not necessarily injured, although strained nearly to their breaking point.

54. A great variation exists in the strength of iron bars which have been cut and welded; while some bear almost as much as the uncut bar, the strength of others is reduced fully a third.

55. The welding of steel bars, owing to their being so easily burned by slightly overheating, is a difficult and uncertain operation.

56. Iron is injured by being brought to a white or welding heat, if not at the same time hammered or rolled

57. The breaking strain is considerably less when the strain is applied suddenly, instead of gradually, though some have imagined that the reverse is the case.

58. The contraction of area is also less when the strain is suddenly applied.

59. The breaking strain is reduced when the iron is frozen; with the strain gradually applied the difference between a frozen and unfrozen bolt is lessened, as the iron is warmed by the drawing out of the specimen.

60. The amount of heat developed is considerable when the specimen is suddenly stretched, as shown in the formation of vapor from the melting of the layer of ice on one of the specimens, and also by the surface of others assuming tints of various shades of blue and orange, not only in steel, but also, although in a less marked degree, in iron.

61. The specific gravity is found generally to indicate pretty correctly the quality of specimens.

62. The density of iron is *decreased* by the process of wire-drawing and by the similar process of cold-rolling, instead of *increased*, as previously imagined.

63. The density in some descriptions of iron is also decreased by additional

hot-rolling in the ordinary way; in others the density is very slightly increased.

64. The density of iron is decreased by being drawn out under a tensile strain, instead of increased, as believed by some.

65. The most highly converted steel does not, as some may suppose, possess the greatest density.

66. In cast steel the density is much greater than in puddled steel, which is even less than in some of the superior descriptions of wrought iron.

IX.

SPECIFICATIONS FOR RAILS.

The German Railroad Union specifies the following in regard to iron rails: —

"The rail top must consist of hard granular iron to a depth of at least $\frac{3}{4}$ of an inch in the finished rail, and the rail base of soft fibrous iron not less than $\frac{9}{16}$ inch thick. The bars laid between the top and bottom slabs must be of such quality as to weld well amongst each other, and to effect a gradual transition of the degree of welding heat, which is lowest in the top slab and highest in the bottom slab. (This clause excludes the use of old rails of uncertain quality.) All mill bars or flats, which are used in piling, must be rolled from hammered blooms. Top slabs, whether made from piles or solid blooms, must be hammered before rolling. The rail piles must be hammered at full welding heat under a steam hammer not less than five tons falling weight; then reheated and rolled."

Specification for Iron Rails for Eastern Counties Railroad, England.

Rails to weigh 65 pounds per yard; 80 per cent. to be 21 feet long, 15 per cent. 18 feet, and the remaining 5 per cent. 15 feet. Any rails which deviate from these lengths more than $\frac{1}{8}$ of an inch will be rejected. The rails to be perfectly true and straight, of uniform section throughout, and the ends to be cut off square. Each rail to be marked on the side with the maker's name, and the date of the year when made. The rails are to be made from a pile 9 inches wide and 9 inches deep, consisting of one bar of iron $9 \times 1\frac{1}{4}$ at the top and bottom. The intermediate bars not to exceed an inch in thickness, and to be alternately 6 inches and 3 inches wide, so as to break joint. The pile is to be rolled at a welding heat into a bloom 5 inches wide and 6 inches deep, which, being again raised to a welding heat, is to be rolled into a rail. The bars, 9″ by $1\frac{1}{4}$″, which form the top and bottom of the pile are to be manufac-

tured from such a mixture of ores (being all mine iron) as shall produce the closest and hardest wrought iron, and shall be drawn from the puddle ball under a hammer which shall be equal to a 5-ton tilt hammer, into a slab 9 inches wide by 2 inches thick, which slab shall be heated sufficiently for its reduction to the required thickness of $1\frac{1}{4}$ inch. The bars of thickness not exceeding 1 inch for forming the central part of the pile are to be manufactured from such a mixture of ores (being all mine iron) as shall produce the closest and toughest wrought iron, and shall be drawn from the puddle ball under a hammer which shall be equal to a 5-ton tilt hammer into a slab or bloom of convenient form, the sectional area of which shall be not less than 20 square inches, which slab or bloom shall be reheated sufficiently for its reduction into bars of the required thickness, not exceeding 1 inch. The use of cinder or cinder pig will not be permitted under any circumstances. The rails are to be dipped while hot in hot linseed oil, and are to be perfectly protected from the weather until this is done. Should any of the rails laminate, break, or otherwise fail, within a period of three years from completion of the order, the company will, at their own expense, take such rails out of the line, and the contractor shall be bound to exchange them for an equal quantity of sound rails, to be delivered when required, free of cost, at the company's grounds.

The reduction of the above pile in rolling is to be from an area of 81 square inches in the pile to an area of 6.8 square inches in the finished rail, or from an area of 12 to 1 very nearly.

X.

SPECIFICATION FOR IRON BRIDGE WORK.

The following general specification is extracted from the circular of Messrs. Clarke, Reeves & Co., of Phœnixville, as applicable to the bridge and viaduct work made at that establishment: —

First. All structures are proportioned to sustain the passage of the heaviest engines and cars in use for coal, freight, or passenger traffic at a speed of not less than 30 miles an hour; viz., two locomotives, coupled, weighing 30 tons on drivers, in a space of 12 feet; total weight of engine and tender, loaded, 65 tons each, and followed by the heaviest cars in use, viz., loaded coal cars, weighing 20 tons each, in 22 feet. The iron-work will be so proportioned that the above loads, in addition to the weights of the structures themselves, shall not strain the iron over 10,000 pounds per square inch of tensile, or 7,500 pounds per square inch of shearing strain, and reducing the strain in compression in proportion to the ratio of length to diameter, by Gordon's formula.

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Second. The iron used under tensile strains shall be of tough and ductile quality, and be capable of sustaining the following tests: —

A round bar, $1\frac{1}{2}$ inches diameter and 12 inches long, shall have an ultimate tensile strength of from 55,000 to 60,000 pounds, and shall show no permanent set under from 25,000 to 30,000 pounds. The reduction of area at the breaking point shall average 25 per cent., and the elongation shall average 15 per cent. It shall bend cold, without sign of fracture, from 90 to 180 degrees.

Third. All workmanship shall be first class. In work having pin connections, all abutting joints shall be planed or turned, and no bars of wrought iron, having an error of over $\frac{1}{64}$ of an inch in length between pin holes, or over $\frac{1}{100}$ of diameter of pin or hole, shall be allowed. In riveted work, all plates and joint plates shall be square and truly dressed, so as to form close joints. Rivet holes shall be spaced accurately and truly opposite. Rivets shall be of the best quality of rivet iron, shall completely fill the holes, and shall have full heads.

Chord-links, main ties, and suspension bolts shall be die-forged without welds. Screw-bars shall have threads enlarged beyond diameter of bar, and shall be fitted with radial nuts and washers.

All bars subject to tensile strains may be tested to 20,000 pounds per square inch, and struck a smart blow with a hammer while under tension; and if any show signs of imperfection, they shall be rejected.

All the iron work shall be painted before leaving the works with one coat of metallic paint and oil. All machine-cut work shall be covered with white lead and tallow before leaving the works.

Fourth. These bridges shall not deflect under the passage of a train of locomotives moving at 30 miles per hour, over $\frac{1}{1200}$ of their length, and shall return to their original camber after the passage of the train.

In addition to the above, the following is given in regard to the preservation of wood and of iron: All of the wood work, track stringers, and corbels, iron platforms, lower lateral rods, suspension bolts, washers, etc., of the Quincy Bridge, were painted with two coats of dark brown mineral paint, from the Brandon Works, Vermont, mixed in linseed oil. The rest of the iron work of the bridge was painted with two coats of the best pure white lead and linseed oil, shaded to a light drab color. The Kansas City Bridge is painted throughout with three heavy coats of a mixture of oil and crushed iron ore, made by the Iron Clad Paint Company, of Cleveland, Ohio, all cracks and weather checks in the timber having been stopped with putty after putting on the first coat. The wooden keys and all joint bearings were painted with the same composition before putting the truss together. Pure red lead has been very generally used for the protection of iron. Some wrought iron rods in a deep well near London, thus painted, were found to be entirely unchanged after 45 years. Oxygen, which is the main cause of rust, is abundant in both water and

air. Quicklime has a great affinity for oxygen, and thus mortar is a sure protection for iron from rust.

In speaking of the anchor masonry of the Niagara Bridge, Mr. Roebling says, "Besides the mechanical protection afforded by the cement grout, I depend principally upon the well-known chemical action of calcareous cement in contact with iron. Oxygen has a greater affinity for lime than for iron. So long, therefore, as the cement will combine with oxygen, or, in other words, has not become completely crystallized, — which is a very slow process inside of heavy masonry, — the iron will be protected. The cement not exposed to the air when setting slowly, has a tendency rather to expand than to contract: but suppose there should be cracks around the anchor bars large enough to admit air and moisture, water will then find its way through those cracks; but on reaching the iron will be more or less impregnated with cement, and thus add another protecting coat."

"On examining recently the anchor-bars of the Monongahela Suspension Bridge, at Pittsburgh, built 16 years ago, I found them perfectly preserved as far as the cement in which they are embedded was removed."

Iron cramps and ties, it is stated by Sir Christopher Wren, taken out from masonry after 400 years, where they had been so bedded as to be perfectly protected from the air, were found to be entirely uninjured, looking as fresh as if just from the forge.

XI.

SPECIFICATION FOR A PASSENGER LOCOMOTIVE ENGINE.

The first specification below was prepared by the late Zerah Colburn. It is given as a comprehensive form for such a paper, rather than as a working specification for the locomotive as now built. The use of steel in place of iron for the furnace, axles, tires, and other members, will suggest itself to any one designing an engine. The second specification has been prepared by the superintendent of the Portland Company's Works, at Portland, Maine, and shows the chief features of the standard eight-wheeled engine for passenger service as now made.

Requirement.

Speed, 20 miles an hour, including stops; fuel, wood; weight of train, 200 tons; maximum grade, 60 feet per mile; sharpest curve, 3°, or 1910 feet radius; rails, 60 pounds per yard, placed on cross ties, two feet from centre to centre; gauge of road, 4 feet $8\frac{1}{2}$ inches.

General Plan and Dimensions.

Outside connections: four 5-feet driving wheels, with best Ames tires, all tires being flanged; level cylinders, 16 inches diameter of bore, and 20 inches stroke; centre bearing truck, with square wrought iron frame, well braced, having inside and outside bearings, and Lightener boxes; four 30-inch best annealed cast iron wheels with chilled tires, the wheels spread 60 inches centre to centre; truck supplied with fore and aft safety chains, and safety beams beneath axles; weight on drivers, 30.000 pounds; on truck, 10.000 pounds. Tender to be mounted on two trucks, each having four 30-inch wheels, spread 54 inches from centre to centre; frames of square iron, well braced, with outside boxes. Tank to hold 2000 gallons.

Detailed Specifications.

Boiler. — Grate, 38 inches wide, and 54 inches long; surface 20 inches above rail; grate bars cast solid for 6 inches of the front end, to be 4 inches deep, and $\frac{3}{4}$ inch thick, placed $\frac{3}{4}$ inch apart in the clear; lower edges chamfered on each side by a chamfer of $\frac{1}{2}$ an inch deep and $\frac{1}{4}$ inch wide; centre of grate bars to be supported by a wrought iron bar 1 inch thick and 4 inches deep, attached as in drawing; outer sides of furnace shell 51 inches wide by 62 inches long; crown 8 feet above rail, to be made of $\frac{3}{8}$ inch iron plates, with a 16-inch necking of angle iron to carry the rear dome; corners to be joined by flanges, rounded to a 4 inch radius; the crown of the shell to be raised 9 inches above the barrel crown, the connection being made by a sloping offset 20 inches long on top; end plates lap-jointed to sides and top, the seams joining the fire box to the waist to be double riveted. Furnace to be made of $\frac{1}{2}$ inch copper plates, $\frac{3}{4}$ inch thick at tubes, lap-jointed, $42\frac{1}{4}$ inches wide, and $51\frac{1}{2}$ inches long inside; side water spaces to be 3 inches clear at bottom, widening (by sloping the sides of the furnace inwards) to 4 inches at the top of the inner box; front spaces 4 inches, rear spaces 4 inches at bottom and 5 at top. Doorway made with a wrought iron ring, fastened with $\frac{5}{8}$ inch rivets; door of $\frac{3}{8}$ inch plate, with $\frac{1}{4}$ inch shield. Furnace joined to shell with $\frac{7}{8}$ inch copper stay-bolts, screwed and riveted at both ends, placed $4\frac{1}{2}$ inches from centre to centre. Eight roof-ribs laid widthwise of the crown of the furnace, being each 6 inches deep and $\frac{3}{4}$ inch thick, double welded at the ends, and riveted at the centre, held down by T head bolts 5 inches between centres; the bars to be raised above the crown sheet by $\frac{3}{8}$ inch thimbles. Necking for dome opening to be made of angle iron, connected with the roof ribs by four $1\frac{1}{4}$ inch stays, placed as in the drawing. The back and tube sheets of the furnace to be flanged over on top, the crown to be flanged downwards on the sides, but not on the back and front. One dome 26 inches diameter, and 24 inches high, to be placed on the

crown of the fire-box shell, the opening of the dome into the boiler being 16 inches in diameter. Lower part of dome of wrought iron, top of cast iron, put on with a ground joint. Furnace and shell to be connected at bottom by a wrought iron bar, 3 inches wide and $2\frac{1}{2}$ inches deep. The whole boiler is to be thoroughly calked inside and out. Barrel of $\frac{1}{4}$ inch best Philadelphia stamped charcoal iron, 44 inches diameter outside, at fire end, and 43 inches at smoke arch: 10 feet long, with 3-inch angle irons at ends. Front dome, 23 inches diameter, of $\frac{1}{2}$ inch plate worked in one piece. End plates of boiler stayed with six 1-inch rods, cottered into blocks, riveted to plates. Barrel plates riveted with $\frac{3}{4}$ inch rivets, and $1\frac{3}{4}$ inch pitch. Smoke arch 2 feet 4 inches long, same diameter as barrel, of $\frac{3}{16}$ inch plates, well riveted and bolted to the angle iron, so as to be easily removed for inside repairs. Front tube sheet $\frac{5}{8}$ inch. Tubes, 140 in number, 10 feet long, 2 inches outside diameter, of No. 9 iron at fire end, and No. 14 at smoke end, with a clearance of half an inch. Chimney of $\frac{1}{4}$ inch iron, 16 inches outside diameter; top, 6 feet 6 inches above crown of barrel, fitted with stack, cone, and spark arrester, as shown in drawings. Ash pan 7 inches deep, riding 6 inches clear of the rail, of $\frac{1}{4}$ inch plate, made with $1\frac{1}{2}$ inch angle iron, and a band on the upper edge, fitted with doors, both in front and behind. Steam pipes of No. 10 copper, 6 inches diameter, running the whole length of the boiler, connected at the domes with 5-inch cast iron stand pipes. Branch pipes in smoke arch, leading to valve chests, of cast iron, 5 inches diameter. Throttle to be in a cast iron chest in the smoke arch, having an area at least as large as that of the steam ports. Changes in the direction of the pipes to be made by curves, and not by angles. Exhaust pipe of No. 10 copper, 5 inches diameter at lower end, fitted with variable blast orifice, ranging from 8 to 4 square inches of area.

Cylinders.—To have a bore of 16 inches diameter, and long enough for a 20-inch stroke, or $28\frac{3}{4}$ inches from outside to outside of ground faces. Casting $\frac{7}{8}$ inch thick, covers $1\frac{1}{8}$ inches thick; the whole placed level, and firmly bolted to the main frame, and to the horizontal truss brace, as shown in the drawing. Valve seat to have steam ports $14 \times 1\frac{3}{8}$ inches; exhaust $14 \times 2\frac{1}{2}$; outside lap $\frac{5}{8}$, and no lap inside; $\frac{1}{16}$ lead on a $4\frac{3}{4}$ inch throw of valve, gradually increasing as the throw is reduced to scant $\frac{5}{16}$. Steam chests bolted to a level face with a ground joint, with $\frac{3}{4}$ inch bolts, and 4 inches pitch. Valve motion, shifting link with lifting shaft, rocker, counter spring, sector, lever, etc., arranged as in the drawings. Four solid eccentrics, of $5\frac{1}{4}$ inches throw, fastened to axle with four square ended set-screws pressing against hardened steel dies cut with sharp grooves on their ends against the axle, the friction of the dies against the axle holding the eccentric in place. Eccentric straps of cast iron, with oil cups cast on, and grooved inside, so as to shut over the eccentric and exclude dust. Link forged, solid and case-hardened, $17 \times 2\frac{1}{2}$ inches inside the slot; thickness of iron all around the slot to be $1\frac{1}{2}$ inches.

and lateral thickness 2 inches. Eccentric rods of $\frac{7}{8}$ inch iron, 3 inches deep, $5\frac{1}{2}$ feet between centres, fastened to the links and to the eccentrics, as in drawings. Link curved to a radius 6 inches less than the distance from centre of driving axle to centre of link at mid-gear. Pistons to be made with one outside composition ring, and two circumferential grooves filled with Babbett metal, and one inside ring of wrought iron; the outside ring to be cut obliquely at one place, with a small wrought iron flap on each edge to prevent leakage of steam at the point of division. Glands of piston and valve-rod stuffing boxes of cast iron, with tight brass or composition bushings.

Frame forged from good scrap, 4×2 inches, the main bar being straight from end to end, with pedestals welded on; the rear end piece to be a heavy forged foot-plate, and the front end an oak beam, 7×14 inches, placed on the flat side; all the pedestals on one side to have adjustable keys. Boiler braces to be flat, $4\frac{1}{2} \times \frac{7}{8}$ inches, with broad palms riveted to the boiler, the attachment at the furnace to be made by an expansion brace. All parts of the frame to be planed true where needed to receive the working parts of the engine.

Wheels, Axles, Springs. — Four cast iron driving wheels, with best flanged Ames tires, 2 inches thick, the diameter with the tire being 5 feet; tires turned to a true cone of $\frac{1}{16}$ inch in a width of 4 inches; the wheels to be truly balanced, to counteract the momentum of the reciprocating machinery. Axles of the best scrap iron, same diameter at centre as at ends, the front driving axle having a diameter of 7, and the rear axle of 6 inches, the bearings being 8 inches long; collars of cast iron held by set screws. Four springs, of 17 steel plates each, $4 \times \frac{3}{8}$ inches, the longest plate being 40 inches; equalizing levers between springs, as shown in drawings. Inside bearing springs of the truck to be hung from an equalizer, the latter bearing upon the axle box.

Slides, Pumps, etc. — Slides, flat wrought iron bars, $3 \times 1\frac{1}{4}$ inches, case hardened. Cross head bearing of cast iron, 16 inches long, and 2 inches thick. Pumps of brass, full stroke, $\frac{5}{16}$ inch thick, with $1\frac{7}{8}$ inch plungers; ram of wrought iron, with an eye fixed on the cross-head, and worked by it. Waterways in body 2 inches, in valves $1\frac{3}{4}$ inches. Three ball valves, with $2\frac{1}{4}$ inch hollow balls, one for suction and two for delivery; pipes $\frac{1}{8}$ inch thick, and 2 inches diameter; suction pipe of iron, delivery pipe of copper; brass cock on delivery worked by a rod from the cab; air chamber on forcing side of pump equal to capacity of barrel, on suction side half the same. Connecting rods, flat bars forged from solid piles without welds. Babbett lined boxes on all stub ends. Straps held on by two bolts each, one key to each bearing. Safety valves, one to be $3\frac{1}{2}$ inches diameter, placed on rear dome, and one 4 inches diameter on forward dome; both to be well fitted, and supplied with the proper levers and spring-balances. Barrel of boiler to be covered with hair felting $\frac{1}{2}$ inch thick, and finished with a Russia iron jacket and brass bands. Cylinders to be protected by a $\frac{1}{2}$ inch felt coat, and cased in brass.

The engine to have all the usual fixtures — bell, whistle, gauges, heater, pet, blow-off, and other cocks, name plate, oil cups, sand box, tools, oil cans, etc. Pilot to be 5 feet long, of flat horizontal wooden bars, $2\frac{1}{2} \times 4$ inches, with a heavy centre-piece, the whole to be well hung and firmly braced. Cab to be neatly built, with a projecting cornice, and windows glazed with plate glass; all the wood-work to be well painted and varnished. Draw-bar to be strongly attached to the frame of the engine, 30 inches above the rail, and connected by a double elliptic spring to the centre beam of the tender.

Tender to hold 2000 gallons; top and side plates $\frac{1}{8}$ inch, bottom plates $\frac{1}{4}$ inch, well riveted and calked inside and out. Brakes to apply from a single wheel to each side of all the wheels; brake blocks hung with safety chains and springs, to carry them away from the wheels. One spring 26 inches long, of ten leaves, $3 \times \frac{5}{16}$ inches, over each wheel. Frame of seasoned oak, 10×4 inches. Centre-beam, 5×20 inches. The whole tender to be thoroughly painted and varnished.

All of the material, both of the engine and tender, to be of the best quality, and all of the construction done in the most thorough and workmanlike manner.

Specification

For an Eight-wheeled Passenger Engine for burning Coal, as built by the Portland Company, at Portland Maine. Gauge of road, 4 feet $8\frac{1}{2}$ inches.

Boiler to be made of the best Pennsylvania iron, $\frac{3}{8}$ inch thick, and double riveted on all the lengthway seams. Diameter of shell, 46 inches at front and $48\frac{1}{4}$ inches at back end. Fire-box 54 inches long inside, 35 inches wide, and 62 inches deep, made of the best quality steel. Tube sheet at furnace of $\frac{7}{16}$ inch steel. Crown bars, nine in number, each made of two bars, $5 \times \frac{5}{8}$ inches, with thimbles one inch thick riveted between them to tie them together, running crosswise over the fire-box and bolted to crown sheet with $\frac{7}{8}$ inch bolts, $4\frac{1}{2}$ inches centre to centre. The crown sheet is also to be supported from the top of the furnace shell by eight stays, $2 \times \frac{5}{8}$ inches. Sides of fire-box, stayed with $\frac{7}{8}$ inch screw bolts, placed 4 inches centre to centre. Longitudinal stays, four in number, $1\frac{1}{4}$ inches diameter, secured to head of boiler by crow-feet. Water-spaces around fire-box, $2\frac{1}{4}$ inches. Dome, 24 inches diameter, and 26 inches high, made of best flange iron. Tubes of iron, 133 in number, 2 inches diameter, and 11 feet long, set with copper ferrules. Lagging of $\frac{7}{8}$ inch white pine, covered with Russia iron, secured by brass bands. Frame, $3\frac{1}{2} \times 3\frac{1}{2}$ inches, forged from the best hammered scrap, with solid jaws, and fitted with cast iron safety wedges, to prevent the frame from taking any of the wear. Cylinders, 16×22. Driving wheels, four in number, 5 feet 7

inches diameter, cast hollow, with steel tires $2\frac{1}{2}$ inches thick. Axle journals, $6\frac{1}{2}$ inches diameter, and $7\frac{1}{2}$ inches long. Centre to centre of driving axles, 84 inches. Centre of forward driving axle to centre of cylinder, 11 feet $5\frac{3}{4}$ inches. Steel slides, $2\frac{3}{4}$ inches wide, $1\frac{5}{16}$ inches thick at ends, and $1\frac{5}{8}$ at middle. Cross heads, rocker boxes, eccentrics, eccentric straps, etc., of best charcoal iron. Rocker shaft of gun iron. Pumps of composition. Links, valve motion, and valve stem of best Low Moor iron, case-hardened. Crank pins of best cast steel. Piston rods of Low Moor iron. Connecting rods of best hammered scrap, fitted with boxes of bronze of the best composition, and lined with Babbett metal. The throttle to be a double-seated balanced valve. Engine truck to have a wrought iron frame, with gun-metal jaws to receive the boxes. Axle journals, $4\frac{1}{2}$ inches diameter, and 7 inches long. Wheels of the best charcoal iron, placed 5 feet 8 inches centre to centre. Cab to be of well seasoned ash, with cherry sash and trimmings. Casings for cylinders and domes, wheel guards, sand box, hand rails, lanterns, brackets, and other accessories, to be made of the best materials, and according to the drawings. The whole weight of the engine to be 31 tons, of which 21 tons are to be upon the drivers. The tender frame is to be of oak, and the tank 17 feet long, 36 inches high, and holding 1900 gallons, of No. 1 charcoal iron, $\frac{3}{16}$ inch thick, for the sides, and $\frac{1}{4}$ inch for the bottom, with two angle irons on all the corners. All of the material for both engine and tender to be of the best quality, and all of the construction to be done in the most thorough and workmanlike manner.

XII.

GENERAL SPECIFICATION FOR A RAILROAD.

THE object of this paper is to define exactly the terms of the contract as regards execution of work. Everything therein should be expressed in a manner so plain as to leave no room for misunderstanding. The following has been prepared from the specifications used in the construction of some of our largest railroads: —

GRADUATION. — LINE.

The centre of the road-bed to conform correctly to the centre line of the railroad, as staked out, or otherwise indicated on the ground, and to its appropriate curvatures and grades, as defined and described by the engineer; and the contractor shall make such deviations from these lines or grades at any time, as the said engineer may require. The road-bed to conform to the cross section

which shall be given or described, or to such other instructions as may be given as hereinafter limited; and the same of the ditches and slopes of the work, and of all operations pertinent to the satisfactory performance of the graduation or masonry on the part or parts of the line contracted for.

Clearing.

The ground forming the base of all embankments, and five feet beyond the foot of the slopes of all embankments, to be cleared as close to the surface as practicable of all timber, saplings, brush, logs, stumps, or other perishable material. The valuable timber to be laid aside, beyond the clearing, as directed by the engineer, the rest to be burned, if this can be done safely, otherwise to be moved beyond the limits of the cleared ground. The ground for ten feet beyond the top lines of all slopes of cuttings shall be cleared in like manner of all timber and saplings. Wherever additional ground has to be taken in widening excavations to obtain materials, or in widening embankments to dispose of surplus material, or in grading for turnouts or depot grounds, an additional amount of ground shall be cleared in like manner, and when directed by the engineer, wherever additional space is required for outside ditching, or for alterations of roads or watercourses, or otherwise.

Grubbing.

All stumps and large roots within ten feet of the grade line shall be grubbed out to the entire width of the work, and moved at least ten feet beyond the slopes. The cost of all clearing and grubbing is included in the price for earthwork, which price is also understood to include all clearing and grubbing necessary in borrowing pits, spoil banks, road crossings, alterations of roads and watercourses, the formation of ditches or otherwise. The necessary clearing and grubbing in all cases to be kept completed five hundred feet in advance of any work in progress.

Mucking.

Wherever mud, muck, or similar soft material occurs in excavations or embankments, within two feet of sub-grade, it shall be removed and replaced by compact earth or gravel.

Grade.

The grade lines on the profiles show the true grade, and correspond with a line two inches below the bottom of the iron rail of the superstructure. What is called sub-grade, corresponds with a line placed eighteen inches below the grade.*

* The distance between "grade" and "sub-grade" depends upon the depth of the ballast, which varies from eighteen inches to two feet.

Width of Road, and Slopes.

The width of roadway, unless otherwise directed, shall be twenty-two feet wide at grade in earth excavations, and eighteen feet wide in rock excavations. Both rock and earth shall be taken out eighteen inches below grade for the entire width of roadway. The bottoming to be replaced by gravel, broken stone, or spawls, in such manner as shall be directed by the engineer, leaving the necessary ditches of the width and depth directed on either side. The contractor will not be paid for any rock excavated beyond the slope lines of one to eight from the required width, or for any earth excavated beyond slope lines of one and one half horizontal to one vertical, unless directed by the engineer to move additional rock or earth.

Blasting.

All blasting shall be done at the risk of the first party, who shall be liable to the second parties, or to the railroad company, for any damages incurred in consequence to dwelling-houses, individuals, or otherwise.

Ditches.

Whenever required, ditches shall be cut along the tops of the slopes, of the form and size, and in the position directed.

Surplus Material.

Whenever the earth or rock required for the adjoining embankments exceeds the amount in the neighboring excavations, the contractor, when required, shall increase the width of said excavations, as directed by the engineer, to a sufficient width for a double track, provided that this additional width shall not be extended so as to produce an average haul of more than eight hundred lineal feet on said borrowed stuff; and whenever the earth or rock to be moved from any cut exceeds in amount the adjoining embankments (unless elsewhere wanted), it shall be applied to widening the embankment to a width for a double track, within the same limits of haul; but for a greater haul than eight hundred feet the contractor shall be paid — of a cent per yard per hundred feet of excess.

Borrow Pits.

Where the excavation does not furnish sufficient material to make the adjoining embankments, borrow pits may be opened; but no earth shall be

deposited in spoil banks, nor borrow pits opened without the knowledge and consent of the superintending engineer, who shall take care that such operations are arranged so as not to damage the road or its slopes, nor to interfere with the widening of the roadbed at a future time for additional tracks.

Material to be saved.

If materials be found in the excavations applicable to useful purposes, such as building-stone, limestone, gravel, minerals, etc., they shall be laid aside in such a place as the engineer may direct, for use, to be applied then or subsequently to the construction of the road, under the conditions of these specifications and of the contract.

Classification of Materials.

Earth —everything except solid and loose rock. *Loose rock* —all bowlders and detached masses of rock measuring over one cubic foot in bulk, and less than five cubic yards. *Solid rock*, includes all rock in ledge which requires drilling and splitting, and all *loose rocks* containing more than five cubic yards.

The prices for excavation include all earth or rock excavated in ditching, bottoming, borrowing, road crossings, alterations of road crossings and water channels, and the construction of temporary roads, provided the average distance hauled on each section be the same as stated on the schedule here annexed; but if the actual average haul on any section is found, on completion, to have been greater or less than the distance stated, a corresponding addition or deduction shall be made of —— per cubic yard per hundred feet which the actual haul exceeds or falls short of that stated.

Embankments.

The embankments to be formed fifteen feet wide on the surface, unless otherwise directed, with slopes of one and one half horizontal to one vertical. Wherever the embankment is formed from ditching on either side, such ditching, and the crest of the slopes thereof, shall in no case approach within six feet, nor within double the depth of ditch, of the foot of the proper embankment slope, allowing always on one side for a double track; and no soft mud or muck shall be allowed to enter the bank. Wherever watercourses or new channels for rivers require to be formed, they shall not approach within once and one half of the depth of such stream, plus twenty-five feet. Care shall be taken in forming embankments to exclude all perishable material.

Subsidence.

To allow for the after settlement of materials on embankments, they shall, when delivered to and accepted by the second parties, be finished to the full width to the following heights above sub-grade, namely: All banks below five feet in height to be finished three inches above sub-grade; banks ten feet high, five inches: twenty feet, six inches; twenty-eight feet, seven inches: thirty-five feet, eight inches; and forty feet nine inches above sub-grade, intermediate heights being in proportion; the engineer having the power to change these proportions at his discretion.

Extra Excavation and Embankment.

Whenever it is considered necessary to increase the width of the roadway for turnouts, water stations, or depot grounds, whether in excavation or embankment, such work shall be done at the contract prices, as may be directed. The opening of foundation pits in simple excavation, where coffer-dams, or such like expedients, are not necessary, and all excavation above the water line in places where such expedients are necessary, shall also be done at such increase or decrease of the contract price as shall be deemed proper by the engineer.

Embanking at Bridges and Culverts.

The contractor for earth-work shall not carry forward in the usual way any embankments within fifty feet of any piece of masonry, finished or in progress (counting from the bottom of the slopes), but shall in every such case have the earth wheeled to the walls or abutments, and carefully rammed to such width and depth, and in such manner as may be directed, when the embankment may be carried on as usual. The expense attendant upon any damage or rebuilding of mason work consequent on neglect of these directions, shall be charged to the account of the first party. In case the mason work shall not be finished when the embankment approaches it, the contractor shall erect a temporary structure to carry over the earth, and proceed with the embankment on the opposite side; and the expense of said structure shall be paid by, and charged to, the contractor for masonry, in case such contractor shall have delayed beyond the proper or required time the construction of the mason work; but if the mason work could not have been ready in season for the bank, then shall the expense belong to the contractor for the earth-work, whose price for graduation is understood to comprehend all such contingencies. For the above work of wheeling and ramming efficiently the earth around any piece of masonry the contractor shall be paid — cents per cubic yard, by the engineer's measurement.

Roads and Watercourses.

The first party is to make good and convenient road crossings wherever directed, and shall also make such alterations of existing roads, or watercourses, or river channels, or such new pieces of these belonging to the section undertaken by him, as may be required, and shall be paid for such work, whether earth, rock, or masonry, the prices, and no more, applicable to this contract; and he shall make such road crossings, or other alterations referred to, at, and within such times, and in such form and manner, as the engineer shall direct; and whenever the operations of the first party interfere with a travelled road, public or private, either by crossing or by making required alterations on it, the first party shall so operate as to afford at all times a safe and free passage to the public travel; and the first party shall be liable for any damage to which the second parties or the railroad company may become lawfully liable by reason of his neglect to maintain a safe and properly protected passage for the current travel.

Ballasting.

Where gravel is used for the ballasting of the road-bed, it shall be of a quality satisfactory to the engineer, and shall be spread upon the road-bed to the width and depth required. When broken stone is used, it shall be of durable quality, and shall be broken so as to pass through a ring of three inches in diameter. The quantity will be measured in the road-bed as finished, and the contractor will be required to keep the ditches trimmed and clear.

Rip-Rap, or Rubble Slopes.

The first party shall distribute rubble stone over the slopes of earth embankments, whenever required to do so, to protect said slopes from the action of water. Such stone to be arranged by competent hands, and laid to such thickness, and with stones of such size, as shall be directed. Where the contractor has rock in the neighboring cuttings which is available, it shall be reserved and applied to this purpose: and when not, good rock shall be obtained where the contractor can conveniently get it.

Measurements.

All earth or rock necessarily moved to complete the grading of this contract according to direction, will be measured in excavation only; and if the contractor (with the consent of the engineer) should find it convenient to waste earth from an excavation, instead of carrying it to its proper embank-

ment, and to borrow earth at some nearer point for said embankment to replace that which was wasted, he shall be paid for the earth from the original excavation in the order of its most economical arrangement for the second parties. All earth moved from borrowing pits shall also be measured in excavation only.

First Class Masonry.

First class masonry will apply to bridge abutments exceeding twenty-five feet in height, to the ring stones of arches, and to the piers of bridges in running water. The stone shall be laid at the rate of one header to two stretchers, disposed so as to make efficient bond. No header to be less than forty inches long, and no stretcher to be less than eighteen inches in width. No stone less than twelve inches in thickness; no stone to have a greater height than width; all stones to be placed upon the natural bed. The masonry throughout to have hammer-dressed beds and joints. Vertical joints to be continued back at least ten inches from the face of the wall. The mortar joints on the face not to exceed one fourth of an inch in thickness. The stone to be laid with regard to breaking joints in the adjoining courses. The stone must be dressed complete before laying, and not be moved after being placed in the mortar. The face will not be tooled, but only roughly hewed, except for one half inch from the beds and joints, where it will be hammered. The ring stones of arches shall have beds to conform to the radius of the arch, with the end joints vertical, and be made to set smoothly on the centring, with the beds with the proper inclination. Each stone must extend through the whole thickness of the arch, and not be less than eight inches thick on the intrados. No spawls or pinners will be admitted. The ring stone shall be dimension work, according to the plans furnished, the beds and joints being truly dressed, but the faces left rough.

All first class work shall be carefully laid in good cement mortar. Each stone before being laid shall be carefully cleaned and moistened; and masonry built in hot weather shall be protected from the sun as fast as laid, by covering with boards. Copings shall be built of stone of equal thickness, neatly dressed and laid.

All first class masonry shall be well pointed with cement pointing.

Second Class Masonry.

To be applied to abutments less than twenty-five feet high, wing and face walls of bridges and culverts, and to piers not in running water, shall consist of stones cut in bed and build to a uniform thickness throughout before being laid, but not hammered; they shall be laid on a level bed, and have vertical joints continued back at right angles at least eight inches from the face of the

wall. The work need not be carried up in regular courses, but shall be well bonded, having one header for every three stretchers, and not more than one third of the stones shall contain less than two cubic feet, or be less than nine inches thick ; and none of that third shall contain less than one and one half cubic feet, or be less than six inches thick. No more small stones shall be used than necessary to make even beds, the whole to be laid in cement mortar and pointed.

Third Class Masonry,

Applicable to culverts, and to the spandrel backing of arches, shall consist of strong and well built rubble masonry, laid dry for culverts, but wet for backing. The culverts to be of such form and dimensions as the engineer may direct. The foundation courses of the side walls to consist of large flat stones, from eight to ten inches in thickness, laid so as to give a solid and regular basis for the side walls. The side walls to be laid with sound stone, and of sufficient size, and with beds having a fair bearing surface and good bond. The covering stone for culverts being not less than ten inches thick for two feet culverts, twelve inches for three feet culverts, and fifteen inches for four feet culverts ; to be free from flaw or defect, and to have a well bedded rest upon each side wall, of not less than twelve inches for two and three feet culverts, and not less than fifteen inches for larger ones. In case such stone cannot be obtained, a dry rubble arch may be thrown instead, well pinned and backed ; but the price for the arch shall not be more than the general price for third class masonry, with an allowance for the centring.

Fourth Class Masonry,

Applicable to cattle-guards, pavements of culverts, and slope and protection walls, shall consist of stones of not less than one cubic foot in contents, so laid and bonded as to give the greatest degree of strength in preference to appearance ; being laid, when directed, with beds perpendicular to the inclined face. Pavements under culverts shall be made by excavating one foot in depth of that part to be paved, which space shall be filled with flat stones one foot wide, set on edge, close together, and made to present an even upper face.

Timber and Plank Foundations.

Timber and plank foundations require the beds to be perfectly well levelled, and timber of such dimensions, and so laid, as shown by the plans ; to be well bedded, and brought to an even and level top surface. The spaces between them to be filled and well rammed with such material as the engineer may direct. On these timbers planks shall be laid, and treenailed or spiked if re-

quired. The materials shall be of quality and shape approved by the engineer, and the price shall be in full for material and labor in laying the whole in a thorough and workmanlike manner.

Piling.

Piling may be used either as bearing piles for foundations, or for piled bridges. In the former case they will be bid for by the running foot driven, and in the latter by the stick of twenty-five feet in length. The piles in either case must be straight, round timber, of a quality approved by the engineer, not less than ten inches in diameter at the small end, barked, and properly banded and pointed for driving. They shall be driven in such places and to such depths as required, and the heads cut off square, or finished with a tenon to receive caps, as may be required. Bearing piles will be cut off so far below the lowest water that any timber foundation laid thereon shall be at all times entirely immersed.

Cement.

Cement, when used, shall be of the best quality, hydraulic, newly manufactured, well housed and packed, and so preserved until required for use; and none shall be used in the work until tested and approved by the engineer.

Cement Mortar.

The proportion of sand and cement for construction shall be one of cement to two of clean, sharp sand, unless in special cases the engineer direct otherwise, for which due allowance shall be made. It shall be used directly after mixing, and none remaining on hand over night shall be remixed.

Lime Mortar.

Lime mortar (which in all cases shall contain cement) will consist, unless otherwise directed, of two parts of best quicklime, one of cement, and five of sand; the ordinary mortar of lime and sand being first properly made, and the cement thrown in and thoroughly mixed immediately before using.

Concrete.

Whenever concrete is required to be used, it shall be formed of clean broken stone, cement, and sharp, clean sand. The stone, which shall be of satisfactory quality, shall be broken so as to pass through a ring three inches in

diameter. The cement and sand shall be thoroughly mixed in the proportions already described for cement mortar. Thus prepared, it shall be carefully mixed with the broken stone in the proportion of one of mortar to two or two and one half of broken stone, as the engineer upon experiment shall determine, and shall be immediately laid carefully in its place and well rammed. The concrete shall be protected on the sides by boards, and be allowed to remain undisturbed after laying until it is properly set; and in special cases the engineer shall direct the mode of application. For the proper preparation and laying of such concrete there shall be paid the price applicable to second class masonry. The contractor shall furnish all tools and plank necessary to the operation.

Pointing.

All masonry in cement or lime will be finished with a good pointing of cement, without extra charge.

Brickwork.

When bricks are required, or allowed to be used, they shall consist of sound, hard-burned brick, laid in cement or common mortar, as directed, and no soft or salmon brick will be admitted; and none but regular bricklayers shall be employed.

Centring and Backing.

The whole top of all arches, whether brick or stone, shall be finished by plastering with a good coat of cement, so as to prevent the percolation of water, and turn it away from the arch. The centring shall be such as the engineer approves in every respect, and shall not be removed until he directs. The cost of backing to be included in the price bid. For arches of more than twenty-feet span, compensation shall be made, at the engineer's estimate, for the extra value and cost of the centring proper for large arches.

General Provision.

The engineer reserves the right to require the whole or any part of the above described work of masonry to be laid in cement, lime mortar, or dry, at his discretion. First and second class masonry, and brickwork, will be bid for at prices for laying in cement, from which will be deducted fifty cents per yard if laid in lime mortar, and one dollar if laid dry. Third and fourth class masonry at prices for laying dry, to which will be added fifty cents per yard if laid in lime mortar, and one dollar if laid in cement.

Scaffolding.

Nothing shall be allowed for workmanship or timber of any scaffolding used in the construction of timber bridges, or in carrying up abutments, piers, coffer-dams, or otherwise. Should the timber used in any coffer-dam be carried away by floods, the renewal of it shall fall upon the first party.

Foundations.

The foundations for all structures shall be executed by the contractor for masonry in such manner and to such depth as to secure a safe and secure foundation, of which the engineer will judge. If a natural foundation cannot be procured at a reasonable depth, then the contractor shall prepare such artificial foundation as the engineer may direct. The stuff moved from the foundations, if of the proper quality, shall be deposited in the adjoining embankment, provided the site for said embankment has been cleared of all perishable material. So much of the stuff as shall not be fit for the embankment, and all roots, stumps, etc., shall be deposited beyond the limits of the clearing, so as not to obstruct roads, watercourses, or ditches.

For the earth moved from such foundations, and for all earth used according to direction, in the construction of coffer-dams, there shall be paid — cents per cubic yard.

Whenever it may be necessary to pump or bale water in the foundations, the contractor shall furnish the pumps or buckets, and all scaffolding and apparatus necessary to work them. He shall be allowed the net cost of all labor employed in the operations of pumping or baling water, and shall make a monthly return to the engineer of the value of such labor, provided that these operations are conducted in an economical manner, with efficient men, pumps, and tools, under the direction and to the satisfaction of the engineer. He shall also be allowed such compensation for the use of the pumps and apparatus, and for superintendence, as the engineer shall judge to be fair and reasonable.

Trestle Work

Includes all wooden structures commonly used as substitutes for abutments and piers, and for farm passes, etc. These shall be built according to the plans furnished, and directions given by the engineer, of sound, durable material, to be approved by him. The price bid shall be by the thousand feet board measure, and will be considered as in full for all material except iron, and for the labor of building and erecting complete. The iron used will be of the best American, and the workmanship of approved quality. The bids will be by the pound, and will cover all cost of material, and the labor

incident to its use. Spikes and nails when used will be furnished by the contractor at cost.

BRIDGING.

Contractors may submit plans for bridging in connection with, or separate from, their bids; but the engineer of the company may reject such plans if he choose, and substitute others, which, if the contractor decline building at the approved prices, may be let to other parties. In every case, the exact manner of building, erecting, adjusting, and finishing bridges, and the determination of the nature and amount of material, will be specified by the engineer. The price bid must be by the running foot of the whole length of bridge, as erected and finished complete.

SUPERSTRUCTURE. — SUBSILLS.

To maintain the track in good adjustment until embankments are settled, subsills will be laid on certain banks, and likewise in cuts where the imperfect nature of the bottoming may, in the opinion of the engineer, render them expedient. These subsills to be fairly bedded in the earth or ballasting, and carefully adjusted and rammed so as to correspond with the grade lines given by the engineer. An additional piece of sill four feet long, shall be laid at each joint of the subsill, either under the sill, or alongside, as may be directed. The sills will be of 3×9 plank, in lengths of twelve, fifteen, eighteen, and twenty-one feet; of which one fourth may be below fifteen, one fourth below eighteen, and one fourth below twenty-one feet. The plank must be square at the ends, and of sound, durable material, and not have more than two inches wane on one end only. There will be about 25,000 feet, board measure, laid per mile where it may be required, and 660 joint sills, 3×9 inches, and four feet long. When the depth of stuff to be moved to admit the subsills exceeds six inches, an allowance shall be made for extra labor, the amount of which shall be noted by the assistants on their receiving notice of such extra labor from the contractor or his agent.

CROSS TIES.

The cross ties shall be of white, black, or yellow oak, burr oak, chestnut, red elm, or other sound timber of suitable character, in the opinion of the engineer, eight feet long, and not more than three inches out of straight, hewn to a smooth surface on two parallel plane faces, six inches apart, the faces being not less than seven inches wide, for at least half of the number, and the remainder not less than six inches wide. The ties shall be carefully and solidly laid on the subsills, or ballasting, or earth previously properly

prepared, so as to give the true planes required by the rails, whether on straight or curved lines. They shall be laid at the rate of eight ties to each eighteen feet rail. All imperfect ties shall be excluded by the track-laying party. The surface of the ties to be faithfully adjusted to the grades given, and to the web of the rail; and the rail to be truly laid, and firmly spiked, so as to correspond neatly to the alignment of the road. There will be about 2500 ties required per mile of road.

Chairs and Joints.

When chairs are used, they shall be such as directed by the engineer, and furnished by the company, and shall be well and accurately placed, and spiked in such manner and position as required, and the largest ties shall be selected for the joints. When the joint is made by fishing, there will be no tie directly under the joint.

Rails.

The rails will weigh about sixty pounds per lineal yard. No rail shall be laid on the tangents which is in any way twisted or bent. It shall be the duty of the first party to correct and make true any crooked rails received by him, also to bend to the proper curve, and in such a manner as not to affect the strength of the bar, all rails laid in curves. Punching of rails, and cutting, will also be done by the contractor.

Track Laying.

The materials composing the track will be furnished by the company, and shall be laid in the best manner, according to the conditions following: The track will be laid on cross ties, and the ties at the proper places on subsills. Where the sills are used, they will be laid with four feet blocks at the joints, and with six feet blocks at the rail joints, the whole being set to their places by stakes, and by the engineer's directions, and mauled down to a perfect bearing, being settled at least half an inch by mauling. The cross ties will be placed uniformly distant, (twenty-eight inches from centre to centre). The iron must be so cut or selected that the joints of the parallel rails shall be within two inches of being opposite to each other; no joint tie being allowed a greater amount of askew than this, whether on tangents or curves. A slip of metal shall be inserted at the rail joints, while laying, to keep the rails apart sufficiently to allow for expansion, which thickness (depending upon the temperature) shall be fixed by the engineer. Two spikes shall be used at each end of each tie, one inside and one outside

of the rail, upon straight lines. Upon curves of less than 1500 feet radius two spikes outside and one spike inside of the rail, at each end of the tie, shall be used. Upon curves the outer rail will be raised to such an amount, depending upon the radius of curvature, as the engineer may direct.

Turnouts.

The contractor to put in such turnouts and sidings, with the necessary frogs and switches, as may be required; the frogs and switches to be firmly and truly placed in position so as to work easily.

Filling and Ditching.

The stuff moved in bedding the sills and ties, to be placed between the latter. The ditches to be properly cleaned out after the track is laid; the filling never to rise higher than the top of the cross ties. Any surplus stuff to be moved out of the cuts, or if on embankments, to be thrown over the bank, leaving the track and road-bed in a neat and workmanlike shape.

Delivery of Materials.

The ties and sills to be delivered at some point on the road, as near as possible to the places where they are to be used, in no case requiring more than one thousand feet of haul; to be so piled as easily to be counted and inspected. The bids for ties will be by the piece, the proposal stating the number and conditions; the sills to be bid for by the thousand, board measure. All material furnished in connection with track laying, to be delivered in such manner and time as to comply, in good season, with the contract for laying the rails.

Measurement of Track.

The measurement of track laid shall include the turnouts, measuring from heel to heel of switch, no extra allowance being made for putting in frogs or switch machinery.

Fencing.

Bids for fencing will be by the running foot or mile, including both sides of the road. Where required, it will consist of posts placed eight feet apart from centre to centre, set three feet into the ground, either by digging or boring, and not by mauling. The posts shall be of oak, elm, chestnut, or other durable wood, not less than eight inches in diameter at the bottom, barked and charred where put into the ground. The boards to be 6×1 inches, and

to square sixteen feet long, to be placed six inches apart, vertically, and fastened to the posts with tenpenny nails at each bearing and breaking joint with each other. The fence will be five bars high, the top of the uppermost being five feet from the ground. In side hill, and on ground liable to slide, particular care shall be taken to place the posts firmly in the ground. At cattle guards, the fence will be turned in to the proper distance, and such arrangement made as to prevent the passage of animals.

General Provisions. — Classification.

The classification of material excavated will be referred to the engineer, in all cases where the nature of the material is questioned, and his judgment taken thereon; also all material used in structures will be submitted to the inspection of the engineer or his assistants.

Quantities and Qualities approximate.

The quantities and qualities of work presented in the schedule are merely approximate, and the information given on the maps and profiles in relation thereto is according to the best present knowledge. The company retains the right to change at any time during the progress of the work the alignment, grades, and width of the road, or any part thereof, and also the limits of the sections; or to alter the character, vary the dimensions, or change the location of structures, or substitute one kind of work or material for another, or to omit entirely, when found necessary, or to require to be built where not now contemplated; and the contractor shall carry into effect all such alterations when required, without the contract prices being thereby affected, unless the aggregate value of all work contemplated by the contract be changed full twenty per cent., in which case a fair allowance, either for the company or the contractor, shall be made by the engineer. In case, however, the aggregate value of the work be changed by over twenty per cent. of the original amount, and the contractor be not satisfied with the altered compensation, then said contractor may throw up said contract, on condition, that within ten days after receiving notice from the engineer of such alteration he give written notice to the engineer or the company of his desire to do so, in which case, as in other cases of throwing up the contract, he shall, as soon as desired, give peaceable possession to the company or their agents, leaving also in their possession any tools or machinery upon which they have advanced anything; and the company may then settle with the contractor on the measure of damages which either shall suffer.

Basis for estimating Effect of Changes.

The basis for estimating any changes as above mentioned is understood to be the schedule exhibited at the letting.

No Liquor, and Good Order.

The contractor shall not sell, or allow to be sold or brought within the limits of his work, any spirituous liquors, and will in every way discountenance their use by persons in his employ. He will do all in his power by his own act, or by assisting the officers of the county, or of the corporation, to maintain the laws and such regulations as conduce to good order and peaceable progress, and prevent encroachment on the rights of persons or property; and he shall discharge from his service, when required by the engineer, any disorderly, dangerous, insubordinate, or incompetent person, and refuse to receive into his employ any who may have been discharged for such cause from other parts of the work.

Monthly Estimates.

Measurements and estimates shall be made by the engineer, once in each month, by means of which may be known approximately the amount of work done, and the contractor shall be entitled to payment, therefor, at such rates below his contract prices as may have been agreed upon by the parties to the contract; it being understood that the contractor has no claim on account of any material not laid in its place in the roadway, or for labor bestowed thereon; and the quantities shall be estimated from the dimensions when so laid, though, on the advice of the engineer, advances may be made on such material when delivered for use, in which case it becomes the property of the company, in the contractor's care and keeping, and he becomes liable for its loss or injury.

Extra Work.

No claim for extra work, or for work not provided for in the contract, shall be allowed, unless a written order to perform such work shall have been given by the engineer; or unless the work be subsequently certified by him, and the certificate produced at the time of demanding the payment of the monthly estimate next after such work shall have been performed.

Sub-Contracts.

The contractor will be required to perform the work himself, and no sub-contracts relieving him from the responsibility of a proper performance of his contract will be permitted, unless by the written consent of the president of the company, and no moneys shall be paid to any such sub-contractor for work or materials, without sufficient authority from the principal contractor.

Time of commencing Work.

On the acceptance of a proposal, the chief engineer will give notice thereof to the person proposing, by letter, directed to his stated address; and in twenty days from the date of such notice, provided there be no impediment on the part of the company, or in twenty days after such impediment is removed if there be any, the work shall be begun with an adequate force, and from that time be prosecuted vigorously until its completion.

How to Progress.

It shall be understood that proper progress is not made, if the amount of work done in each month is not in due proportion to the total amount to be done up to the time fixed for completion by the contract; in which case the engineer shall call the attention of the contractor (or whoever may be in charge of the work if the contractor be absent) to the fact, and state to him what additional exertion is necessary to be made, and what further force is required, in such reasonable time as may be prescribed.

Putting on more Force.

In default of the contractor's making such additional exertion, and supplying such force, the chief engineer, or president of the company, may have such force sent to the work, and the necessary buildings may be erected to receive them, at the contractor's charge and expense, who shall receive the said force in his employ, and work it at whatever price it may have been found necessary to employ it, without diminishing the previous force of the work, and regarding, always, such extra force as if employed by himself.

Causes for Detention.

There shall be no claim for detention on account of work not being laid out, unless a written notice, three days in advance, that it is required, shall have been given to the engineer; and the damage for such detention shall be

estimated by the engineer. The right of way shall be furnished by the company; but if it fail to do so for any particular place, damages for detention shall not be claimed unless the contractor be detained full twenty days after he shall have given written notice to the engineer, of his wish to commence work at such place. Then the engineer may either estimate to him the amount of damage which he shall take as satisfactory, or he may extend the time of the completion of such work by as many days beyond the contract time as the contractor is detained beyond the twenty days following his notice to the engineer.

The Engineer.

In all cases where the word "engineer" is used, the engineer in charge of construction is meant; but the directions of any subordinate engineer shall be obeyed when given in regard to any of the ordinary operations, or where they are evidently in accordance with the specifications, or when transmitting the orders of his superiors. In other cases they may be referred to the resident engineer, and finally to the chief engineer, he being the authorized officer at the time acting in that capacity.

Contractor.

The word "contractor" applies to and includes all persons contracting jointly, any one of whom shall be considered the authorized agent for and in behalf of his associates, and empowered to receipt payment of moneys, receive and act upon orders.

XIII.

RESISTANCE FROM CURVES.

The first important experiments made in the United States to determine the resistance to traction upon railways, were those carried out by B. H. Latrobe, upon the Baltimore and Ohio Railroad, in April and May, 1844. The principal object was to test the merits of three different patterns of coal cars, with four, six, and eight wheels respectively. The trials were made about a quarter of a mile from the west gate of the Mount Clare Depot, and at points just east and west of the crossing of the Washington Branch, at the north end of the Thomas Viaduct. The amount of traction was ascertained by weights suspended in a scale dish weighing 40 pounds, and hanging by a rope over a pulley placed upon the top of a frame resting on a light four-wheeled car, which was pushed by men before the car experimented upon, which followed at a uniform velocity. The speed varied from $1\frac{3}{4}$ to $3\frac{1}{2}$ miles an hour. The num-

ber of experiments was 36, and the results classified according to the kind of cars, and averaged, are given in the following table. The general average is deduced from a combination of the first averages, and is thus somewhat different from the general average that would be deduced by throwing all the experiments together without classification.

Description of the kind of Car.	Weight in Tons of 2240 Pounds			Traction in Pounds per Gross Ton	
	Car	Load.	Total.	On Straight Line.	On Curve 400′ Radius.
Four-wheel coal car, .	1.74	4.76	6.49	4.82	8.32
Four-wheel box car, .	1.10	1.75	2.85	5.00	9.08
Six-wheel coal car, . .	3.02	5.24	8.26	8.47	14.70
Six-wheel coal car, . .	2.59	6.87	9.46	11.21	20.63
Six-wheel coal car, . .	2.59	6.83	9.42	8.60	18.26
Six-wheel coal car, . .	2.59	4.47	7.08	5.17	13.14
Six-wheel coal car, . .	2.48	6.55	9.03	4.29	8.22
Eight-wheel coal car, .	2.72	7.59	10.31	6.84	10.09
Eight-wheel house car,	4.26	4.01	8.28	9.10	14.50
Eight-wheel house car,	4.76	5.03	9.80	9.26	19.39
Eight-wheel both kinds,	4.51	4.52	9.04	9.18	16.94
Eight-wheel pass. car,	7.90	. . .	7.90	5.27	19.00

From the above figures the following averages are deduced: —

Description of the kind of Car.	Straight Line.	Curve 400′ Radius.
Four-wheel cars, ...	4.91	8.70
Six-wheel cars, ...	7.55	14.99
Eight-wheel cars, ...	7.93	15.98
General average, ...	6.80	13.22

Or, as a general result, it may be stated that the resistance upon a curve of 400 feet radius is double that upon a straight line.

The next detailed results are furnished by an extended experiment upon

the largest scale carried out upon the New York and Erie Railroad, in August and September, 1855, by the late Zerah Colburn, under the direction of D. C. McCallum, then general superintendent of that road, the object being to determine the relative power required upon the several divisions of the road for the transportation of heavy freight. For this purpose a single engine was run the entire distance from Dunkirk to Piermont, 445 miles, with trains varying to suit the ruling grades of the several divisions. The engine employed had four coupled drivers and a truck. The total weight was 66,050 pounds. Weight on drivers, 40,050 pounds. Cylinders, 17 × 24. Driving wheels, 5 feet. Maximum pressure of steam on pistons, without slipping the wheels, 140 pounds, or, deducting the atmospheric pressure, $125\frac{3}{10}$ pounds effective pressure per square inch. The tractive power of the engine was thus: —

$$\frac{125\frac{3}{10} \times 17 \times 17 \times 24}{60} = 14{,}485 \text{ pounds.}$$

The engine and tender were moved with slightly accelerated motion, on a level, under an effective pressure of 3 pounds. Their friction, therefore, without any load attached was, —

$$\frac{3 \times 17 \times 17 \times 24}{60} = 347 \text{ pounds.}$$

It has been customary to estimate the friction of cars, with wheels 30 inches, and journals 3 inches diameter, at about 7 pounds per ton, or 8 pounds per ton for 33-inch wheels and $3\frac{7}{8}$ inch journals; but the experiments show plainly that the friction of the loaded cars did not exceed $4\frac{1}{2}$ to 5 pounds per ton. It has also been usual to estimate the additional friction of the engine in consequence of its load at 1 pound per ton of its load on a level. This item will of course be reduced as the friction of the cars is reduced. After a careful examination and comparison of the loads moved upon the ruling grades and curves of various sections of the road, it was assumed that the friction of the cars was $4\frac{1}{2}$ pounds per ton of 2000 pounds. The resistance of curves, $\frac{1}{2}$ pound per ton per degree of curvature per 100 feet; and the additional friction of the engine $\frac{1}{2}$ pound per ton of load on a level and straight line, or its equivalent. The weight of the engine on its drivers being 40,050 pounds, and the traction 14,485 pounds, the adhesion was $\frac{14485}{40050}$, or $\frac{36}{100}$ of the insistent weight. The tender with wood and water weighed 40,240 pounds.

A train of 100 loaded cars, weighing 3,423,150 pounds, making the total weight of engine, tender, and train 3,529,440 pounds, or 1765 tons very nearly, was taken over a mile of road, on an ascent of 6.14 feet, and a curve of 1°, or 5730 feet radius, in $11\frac{1}{2}$ minutes. The preceding mile being also on a uniform ascending grade of 6 feet, no advantage was obtained by momentum previously acquired. The resistances overcome in this case were as follows: —

Friction of engine and tender, ...	347	pounds.
Friction of cars, $1,711\frac{575}{1000}$ tons, at $4\frac{1}{2}$ pounds, .	7,702	"
Gravity of engine and train, $\frac{3.529.440 \times 6.14}{5280}$, . . .	4,104	"
Resistance of curve, $1,765 \times \frac{1}{2}$, ...	882	"
Additional friction, $\frac{1}{2}\left(\frac{4,104 + 882}{4\frac{1}{2}} + 1,711\frac{575}{1000}\right)$, .	1,410	"
Total resistance, ...	14,445	"

or 40 pounds less than the estimated traction.

A train of 22 cars, weighing 753,082 pounds, or $376\frac{541}{1000}$ tons, and with engine and tender weighing 859,372 pounds, or $429\frac{686}{1000}$ tons, was taken up a mile of $60\frac{1}{2}$ feet ascending grade, through a curve of 5°, or 1146 feet radius, in $6\frac{1}{2}$ minutes. The resistances in this case were as follows : —

Friction of engine and tender, ...	347	pounds.
Friction of cars, $376\frac{1}{2}$ tons, at $4\frac{1}{2}$ pounds, . . .	1,694	"
Gravity of engine and train, $\frac{859\ 372 \times 60.5}{5280}$, . . .	9,847	"
Resistance of curve, $429\frac{686}{1000} \times 2\frac{1}{2}$, ...	1,074	"
Additional friction, $\frac{1}{2}\left(\frac{9,847 + 1,074}{4\frac{1}{2}} + 376\frac{1}{2}\right)$, . . .	1,401	"
Total resistance, ...	14,363	"

or 122 pounds less than the maximum tractive power of the engine under an effective steam pressure of $125\frac{3}{10}$ pounds per square inch.

A train of 25 loaded cars, weighing 870,250 pounds, or $435\frac{1}{8}$ tons, and with engine and tender 976,540 pounds, or $488\frac{77}{100}$ tons, was drawn up a mile of 52 feet grade through a 5° curve, or 1146 feet radius, in 9 minutes, the resistance thus being, —

Friction of engine and tender, ...	347	pounds.
Friction of cars, $435\frac{1}{8}$ tons, at $4\frac{1}{2}$ pounds, . . .	1,958	"
Gravity of engine and train, $\frac{976,540 \times 52}{5280}$, ...	9,618	"
Resistance of curve, $488\frac{77}{100} \times 2\frac{1}{2}$, ...	1,220	"
Additional friction, $\frac{1}{2}\left(\frac{9,618 + 1,220}{4\frac{1}{2}} + 435\frac{1}{8},\right)$ . . .	1,422	"
Total resistance, ...	14,565	"

being an over estimate of resistances, or an under estimate of traction, of 80 pounds.

A train of 23 loaded cars, weighing 800,330 pounds, or $400\frac{165}{1000}$ tons, and with engine and tender 906,620 pounds, or $453\frac{31}{100}$ tons, was taken up a mile of 60 feet ascending grade, through 2900 feet of $3\frac{1}{2}$ degree curve or radius, 1637 feet in 5 minutes, the resistance being, —

Friction of engine and tender, ...	347	pounds.
Friction of cars, $400\frac{1}{6}$ tons, at $4\frac{1}{2}$ pounds, . . .	1,800	"
Gravity of engine and train, $\frac{906,620 \times 60}{5280}$, ...	10,302	"
Resistance of curve, $453\frac{31}{100} \times 1\frac{3}{4}$, ...	793	"
Additional friction, $\frac{1}{2}\left(\frac{10,302 + 793}{4\frac{1}{2}} + 400\right)$, . . .	1,433	"
Total resistance, ...	14,675	"

or 190 pounds over estimate of resistances.

A train of 24 cars, weighing 821,544 pounds, or $410\frac{772}{1000}$ tons, and with engine and tender 927,834 pounds, or $463\frac{917}{1000}$ tons, was taken up a mile of 60 feet grade, without curvature, in $5\frac{1}{2}$ minutes. The resistance was thus: —

Friction of engine and tender, ...	347	pounds.
Friction of cars, $410\frac{8}{10}$ tons, at $4\frac{1}{2}$ pounds, . .	1,848	"
Gravity, $\frac{927,834 \times 60}{5280}$, ...	10,543	"
Additional friction, $\frac{1}{2}\left(\frac{10,543}{4\frac{1}{2}} + 410\frac{8}{10}\right)$, ...	1,377	"
Total resistance, ...	14,115	"

the resistance being 370 pounds less than the traction.

The same train was taken the next mile on a grade of 58 feet, through a curve of $3\frac{1}{2}°$, or 1637 feet radius, for 1500 feet, in $8\frac{1}{2}$ minutes, the resistance being, —

Friction of engine and tender, ...	347	pounds.
Friction of cars, $410\frac{8}{10} \times 4\frac{1}{2}$.	1,848	"
Gravity, $\frac{927,834 \times 58}{5280}$, ...	10,192	"
Resistance of curve, $463\frac{9}{10} \times 1\frac{3}{4}$, ...	812	"
Additional friction, $\frac{1}{2}\left(\frac{10,192 + 812}{4\frac{1}{2}} + 410\frac{8}{10}\right)$, . .	1,428	"
Total resistance, ...	14,627	"

or an over estimate of resistance of 142 pounds.

The average of these six experiments shew an estimated resistance of 14,465 pounds, or 20 pounds less than the computed maximum power of the engine, with the steam gauge at 140 pounds pressure.

The resistances in these experiments were unusually small, or else the adhesion was unusually large. In ordinary practice one fifth of the weight upon the drivers is a very favorable result, one seventh being more common.

XIV.

THE RELATION OF GRADES WITH AND AGAINST TRAFFIC.

In Chapter III. reference has been made to the relative grades upon the two sides of the summit of the Portland and Ogdensburg Railway, at the Crawford Notch of the White Mountains. Assuming, says Mr. Latrobe, in his report upon that location, 42 tons on the drivers of an engine, and 23 tons for the weight of the tender with wood and water, making 65 tons in all, and a frictional resistance, including that from curvature, for the tender and cars following, of 12 pounds per ton of 2000 pounds, and the adhesion being $\frac{1}{7}$th, or 12,000 pounds, which is as much as can be safely counted upon, the gross loads that could be drawn up different grades would be, —

116 feet grade, 158 tons of 2000 pounds.
150 feet grade, 117 tons of 2000 pounds.
200 feet grade, $77\frac{1}{2}$ tons of 2000 pounds.

If we assume three tons of freight to move east to one ton west, and if we allow each car to carry its own weight of freight, then if the grade on the Saco (eastern) slope be 116 feet, that on the Ammonoosuc (western) should not exceed 78 feet; with a grade upon the east of 150 feet, that on the west may be 106; and with a grade of 200 feet on the east side, we may employ one of 150 feet upon the west.

The reason why, as the grade is increased upon the eastern side it may also be increased on the western, is, that the cars which are assumed (and correctly) to be the same in number and weight, irrespective of their freight, in both directions, constitute a constant quantity together with the engine and tender, which bears a larger and larger proportion as compared with the freight to the weight of the whole train, and consequently causes the aggregate weights to approach equality more and more nearly in each direction of movement.

If, says Mr. Haupt, at the conclusion of a paper upon the effect of grades (Van Nostrand's Magazine for 1871), the gross load of a train on a grade of 30 feet be 541 tons, the engine and tender being 63 tons, the cars and contents will weigh 478 tons; or, if 18,000 pounds, be allowed for each car, and 22,000 pounds for load, the number of cars will be 27, and the net load 261 tons, and weight of cars 217 tons.

If the return cars were only one fourth loaded, the gross weight of the trains would be 345 tons.

The inclination that would employ the full power of the engine in hauling 345 tons would be 45 feet. The inclination that would employ the full power of an assistant engine in hauling a gross load of 345 tons would be 102 feet; but allowance must be made for the weight of the assistant engine.

XV.

THE CAPACITY OF RAILWAYS AS AFFECTED BY GRADES.

Mr. E. C. Rice, chief engineer of the St. Louis and South-Eastern Railroad Company, in a valuable report, makes the following remarks concerning the reduction of the first cost of railways: —

Every road should be located with reference to eventually having as light grades and curves as practicable, the maximum grade being adopted against the lightest trade; but the road should be built and the rolling stock adapted to the demands of the traffic for the first few years.

When the business of a road at first will be small, and it is necessary to construct it as cheaply as possible, instead of making a narrow gauge, it is better to economize as follows: —

1. Make a careful location with reference to the maximum grade which has been judiciously adopted for the whole road, or each working division, in view of the cost and the demands of business; then locate and construct a cheap temporary parallel line where heavy, costly work is required to build the permanent road, as near as possible to the true location, using, if necessary, heavy, undulating grades to make the work as cheap as practicable at first, and to save time in building, and thus have a cheap road-bed, and most of the distance the true location and grades.

As soon after the road is built, as may be required, the permanent road may be completed.

2. By the general use of undulating grades, without increasing the maximum grade, or decreasing the capacity of the road.

3. By using light iron and rolling stock.

The gauge of a road cannot be widened without great cost and loss, but the grades can be made lighter, and the strength and weight of rolling stock and iron can be increased, as the business of the road may require, without loss: for, as iron and rolling stock wear out, they can be replaced with heavier, changing gradually as the business requires.

It is economy to use *some* light rolling stock and iron on any road.

When roads are not worked up to their capacity, the effect of grades on the cost of transportation is chargeable to the cost of running the engines, and the extra wear and tear of road and machinery; all other cost being in proportion to the tonnage.

But when a road is to be worked to its full capacity, its greatest capacity is an important element in the calculation.

It is evident that the measure of the capacity of a railroad is the number and weight of trains which may be passed over it; and if, from adverse grades,

or other causes, the trains are reduced one half in weight, the capacity of the road must be reduced in the same ratio, and the expenses per mile of road are consequently at least doubled per ton carried.

Very few roads are worked to their full capacity, and lighter engines and lighter trains may be run, and at much lower speed.

No two roads may need exactly the same weight of engines and cars for true economy.

The cost of wear and tear of machinery and road is nearly the same whether heavy machinery is used and actually required on account of grades and the demands of business, or whether it is used without any reason, and through ignorance of the true economy of operating.

In building a road, not only the first cost of road and stock should be taken into account, but the cost of operating. Grade reduces the capacity of a road, and increases the cost of transportation, far more than the owners of the roads realize. The effect of grades can be readily established by calculations showing the different loads that engines of given power can haul over different grades.

The following are some of the results of the workings of the Philadelphia and Reading Railroad for five years, taken from Mr. Steele's report of 1856, with his calculations based on the actual cost of transporting 9,000,000 tons of coal 93 miles over virtually a level road.

Grades of 25 feet per mile against the trade reduce the capacity (a level road being taken at 4,000,000 tons per annum) to 1,900.000. Grades of 50 feet reduce it to 1,200,000. The average load of a locomotive is taken at 437 tons, which is the result obtained upon the above road.

The average work of the Reading Railway, for five years, was 1,816,813 tons per annum, or something less than half its ultimate capacity; the figures are, therefore, due to half the capacity of any other road which may be under consideration, and all less quantities than half capacity will be carried at greater cost.

Mr. Steele gives the following as the cost of transporting coal on roads of various grades, exclusive of drawbacks, or of interest on capital: —

Level roads — average net load 437.2 tons.

	Tons	No. Trains.	Cost per Ton per Mile.
Capacity,	4,000,000	7,149	65-100 cents.
½ do.	2,000,000	4,574	69-100 "
¼ do.	1,000,000	2,287	75-100 "

Grade of 22 feet per mile — net load 233.5 tons.

	Tons	No. Trains.	Cost per Ton per Mile.
Capacity,	2,113,449	9,149	90-100 "
½ do.	1,056,724	4,574	92-100 "
¼ do.	528,362	2,287	104-100 "

Grade of 25 feet per mile — net load 205.7 tons.

	Tons.	No. Trains.	Cost per Ton per Mile.
Capacity	1,881.951	9.149	92-100 cents.
½ do.	940.975	4.574	99-100 "
¼ do.	478.488	2,287	113-100 "

Grade of 50 feet per mile — net load 128.8 tons.

Capacity	1,178.392	9,149	131-100 "
½ do.	589.196	4.574	142-100 "
¼ do.	294.598	2,287	164-100 "

Grade of 55 feet per mile — net load 119.1 tons.

Capacity	1.089.646	9.149	138-100 "
½ do.	544.823	4.574	150-100 "
¼ do.	272,411	2,287	173-100 "

The capacity referred to is for double-track roads.

If it were possible to pass trains so quickly in opposite directions over single-track roads, their capacity would be half that of the double track, but, as it is, it has been taken at one fourth.

We occasionally see it gravely asserted that grades are of small importance on railroads, but we have been reviewing an extended system, in which the capacity varies from 500,000 to 4,000,000 of tons per annum, and this variation is produced by grades.

If we compare two roads, each 100 miles long, the one having grades of 25 feet per mile and the other level, and the demands of transportation on each amount to 2,000,000 of tons per annum, the difference in favor of the level road is $600,000, or the interest on $10,000,000.

But the level road can do twice as much work as the other, and that with the same amount of motive power, so that whilst the graded road is carrying its 2,000,000 of tons for net cost, the level road can, at the same prices, realize on its 4,000,000 of tons a profit of $1,200,000, or the interest on $20,000,000.

Mr. Steele's calculations are based upon the transportation of coal, which can be carried at less rates per ton per mile than general merchandise; but the relative difference of cost remains, and it is applicable to some considerable extent to the conveyance of passengers.

Mr. Steele's tabular statement of the effect of various grades on the load of locomotive engines of 43½ tons weight, including tender, fuel, etc., is as follows: —

EFFECT OF GRADES ON THE LOAD HAULED.

Grade per Mile.	Gross Load including Engine, Tender, and Cars.	Gross Load exclusive of Engine and Tender.	Net Load.	Ultimate Capacity of a Double Track Railroad.	Ultimate Capacity of a Single Track Railroad
LEVEL	767.5	725.0	437.2	4,000,000	1,000,000
22 feet.	429.8	387.3	233.2	2,136,322	528,362
25 "	383.7	341.2	205.7	1,881,951	470,488
50 "	255.8	213.3	128.8	1,178,392	294,598
55 "	239.8	197.3	119.1	1,089,646	272,411
75 "	191.8	149.3	90.1	824,325	206,081
100 "	153.5	111.0	67.0	612,986	153,246
125 "	127.9	85.4	49.5	452,811	113,202
150 "	109.6	67.1	38.9	355,845	88,961
175 "	95.9	53.4	30.9	282,662	70,665
200 "	85.3	42.8	24.7	225,946	56,486
225 "	76.7	34.2	19.7	180,207	45,052
250 "	69.7	27.2	15.7	143,615	35,904
275 "	63.9	21.4	12.4	113,428	28,357
300 "	59.0	16.5	9.5	86,899	21,725

The average cash cost of a single track on western railroads cannot exceed $25,000 per mile, and to build an additional track would cost, say, $18,000 per mile. To reduce the grades one half would not exceed $10,000 per mile on many of the roads, and would not exceed $18,000 on any except at a very few points. As grades have so great an effect on the capacity of roads and the cost of transportation, the true economy is to increase the capacity and reduce the grades as much as practicable before multiplying tracks.

The question for any road worked to full capacity is the economical size of engines and trains before reducing the grades or multiplying tracks.

When the traffic is uniformly lighter in one direction than another, the grades can be made steeper against that trade.

It is very important, however, to make the prices of transportation with reference to securing as far as possible an equal amount of traffic in each direction, so as to reduce the dead weight carried.

Mr. A. J. Cassatt, general superintendent of the Pennsylvania Railroad, furnishes the following facts regarding the actual load hauled by engines over different grades upon that railway: —

A standard 10-wheeled freight engine, with 3 pairs of $4\frac{1}{2}$ feet drivers, with average coal and water, weighs . . . . 75,500 pounds.

The weight on the drivers is, ... 53,000 "

The weight of tender, with wood and water, is . . . 50,000 "

Such an engine will haul on a moderately straight and level road 50 loaded cars, of 40,000 pounds each, or a gross load of, ... 1,062 tons.

On a grade of 10 feet per mile, 43 cars, or a gross load of, 922 tons.

On a grade of 26 feet per mile, 35 cars, or " " 762 "

On a grade of 52.8 feet per mile, 17 cars, or " " 402 "

On a grade of 96 feet per mile, 11 cars, or " " 282 "

And Mr. Cassatt states, that the engine would work easier with 50 cars on the level than in either of the other cases, and with most difficulty in the last.

Herman J. Lombaert, Esq., vice-president, and former superintendent of the same road, gives as a full average load, for actual work, in the usual conditions of the rail, the following: —

52.8 feet grade, 16 cars, gross load of engine, ... 382 tons.

10 feet grade, 40 cars, " " " . . . . . 862 "

the latter results being somewhat less than those given by Mr. Cassatt.

As remarked in Chapter III., the cost of transport upon railways is not proportioned to the power expended in drawing the trains, and is thus not directly proportional to the resistance produced by grades. The error of supposing the cost of transport upon railways to be directly as the power expended, frequently appears in discussions upon this subject. Thus, in the report upon canals, by the State Engineer of New York, for 1869, we find the statement below: —

"The following calculation shows the net load, in tons of coal, a first class engine will haul over different grades at a speed of 15 miles an hour, and also the cost per ton of transporting the same: —

Grade in Feet per Mile.	Net Load in Tons (2240 lbs)	Number of Cars.	Cost in Mills per Ton per Mile.
On a level, ...	690	69	4.40
Grade 10 feet per mile.	480	48	6 40
Grade 20 feet per mile, ...	350	35	8 70
Grade 30 feet per mile, ...	250	25	12.10
Grade 40 feet per mile.	230	23	13.40
Grade 50 feet per mile.	200	20	15.60
Grade 60 feet per mile.	170	17	18.30
Grade 70 feet per mile.	154	15	20 20
Grade 80 feet per mile.	130	13	24.00"

The conclusions in the last column will appear sufficiently absurd to any one who has paid attention to the working of railways.

XVI.

ON THE ADAPTATION OF LOCOMOTIVE POWER.

In 1866, B. H. Latrobe, Esq., made a report to the president of the North Missouri Railroad Company, in which he recommends two classes of engines.

1st. The common pattern, for passenger and fast freight, and stock.

2d. An 8-wheeled coupled engine, say with 44″ drivers, and cylinders large enough not to need over 100 pounds of steam effective pressure to slip the wheels upon a good rail, and a boiler large enough to make steam at 12 miles per hour. The following is taken from Mr. Latrobe's report: —

"As to the weights of the two classes of engines, this must depend somewhat upon what weight you can afford to give to your rail. For a fifty pound rail I would not use engines of more than twenty-five tons (2000 pounds) weight for passenger, and thirty tons for freight, and for a sixty pound rail, thirty and thirty-five tons respectively. These engines will draw up your steepest grades the heaviest passenger trains you are likely to have for some time, and freight trains of from three hundred and fifty to four hundred tons gross (that is, cars and load together), according to the weight of rail and engine you may use. These would be the performances of the machine when in perfect condition, although a less average duty would be realized from them in their ordinary every-day working order.

"In estimating as above the performance of engines, I have assumed as your steepest grade, one foot in one hundred, or 52 feet 8 tenths per mile, and the most abrupt curve, occurring on that grade, 3 degrees, or 1910 feet radius.

"The speed of express passenger trains may average twenty-five miles per hour, inclusive of stops, reaching thirty or even thirty-five miles per hour on suitable parts of the road, and occasionally, to make up lost time, forty miles per hour would not be attended with danger, if track and machinery are in good order. The speed of fast freight and stock trains may be fifteen miles per hour, including stops, with an extreme of twenty miles per hour running velocity at times. Trains with ordinary freight, and especially if carrying coal, ore, stone, lumber, and other heavy articles, should not average more than eight miles per hour, nor exceed twelve miles per hour under any circumstances.

"I am aware that in recommending the connection of all the wheels of locomotives for freight trains, I differ from the popular idea upon most of the railways of the country, on which the engine with four or six drivers, and a truck, is mostly preferred. Until I can discover more clearly, however, the

principle on which such preference is founded, I must adhere to my own views upon the subject. The usual reason given for coupling the power with only a part of the weight of the machine, which weight alone makes the power effective, is to save the track. However good this reason may be for high velocities, it does not apply to slow speeds with any material force, or at least not with sufficient to balance the advantage of increased effective power in the engine. If by uncoupling your front or your two front pairs of wheels you lose a fourth or a third part of your adhesion, you must add a fourth or a third to the number of your engines, and in that proportion to the number of miles run to do the same work, thus increasing in nearly the same ratio the wear and tear of your track. It can scarcely be imagined that the increased smoothness of motion due to the substitution of the truck for the drivers will compensate for this increase, not only of wear and tear of road, but also of the other locomotive expenses.

"I know, however, by experience, how difficult it is to settle such points by mere force of argument on general principles. Facts are the only arbitrators we can successfully appeal to, and I would therefore adduce a few to sustain my position. If the use of the connected engine did, in fact, produce greater wear and tear of track, we should see the result in the expense of maintaining it on those roads on which that kind of locomotive is most employed. Let us look, then, at the working of the four great east and west lines, viz., the New York Central, the Erie, the Pennsylvania Central, and the Baltimore and Ohio Railroads; and to bring the last fairly into the comparison, we will select the year 1860, before the war interrupted its business and inflated prices upon all the roads.

"Now, upon the New York Central there were no connected engines excepting such as may have been used at the stations in shifting trains; on the Erie, there were but six out of two hundred and twenty; on the Pennsylvania Central, nineteen out of two hundred and eleven; on the Baltimore and Ohio, one hundred and forty-seven out of two hundred and nineteen. From this it will be seen that upon all but the last of the four roads the transportation of freight was effected entirely, or almost entirely, by truck engines; while upon the last it was done almost altogether by connected engines; for the nineteen connected engines on the Pennsylvania Central and the six on the Erie, were used only as assistants on the heaviest grades of those roads, while on the Baltimore and Ohio, after deducting forty-seven truck engines employed in the passenger business, there remained only twenty-five truck engines in the freight business, and they so inferior in weight and power to the one hundred and forty-nine connected engines, that these last may well be said to have done the whole freighting of the road. How, then, does the expense of maintenance of road and machinery compare on the four roads in that year (1860)? The published reports of the several companies show the following results: —

NAME OF ROAD.	No. of Miles Run.	Track Repairs per Mile run.	Engine Repairs per Mile run.	Engine and Car Repairs per Mile run.	Total Cost of Working Road per Mile run.
		CTS.	CTS.	CTS.	CTS.
New York Central, . .	4.493.213	19.80	9.00	17.90	95.22
Erie, ...	3.474.917	24.13	9.01	20.70	94.60
Pennsylvania Central, .	3.633.442	21.38	7.69	15.90	99.26
Baltimore and Ohio, . .	3.831.295	15.90	6.78	15.44	51.79

"Had the Reading Railroad been brought in, it would show its track repairs as 11.22 cents per mile run, and its engine repairs as 8.55 cents, and yet the Reading road does its entire slow freight and coal business with connected engines.

"The year 1860 exhibits a fair average of preceding and succeeding years, and may, therefore, be safely taken as showing the effect of using this class of locomotives in the transportation of heavy freight.

"An inspection of the above comparative statement shows that the repairs of track as well as of engines alone, and of engines and cars together, and also of the total cost of operating the road, are less upon the Baltimore and Ohio Railroad than upon any of the three others, notwithstanding that the grades and curves of that road are stronger than theirs, which, if the above mentioned objection to the connected engine were valid, should lead to a wholly different result. In truth, the connected engine, whose front and back wheels only are flanged, and also closer together than those of the truck engines, turn curves at slow speed more easily than the latter in consequence of these features in their construction.

"The use of the connected engine, and of heavy and powerful engines generally, has been considered by many as advisable only on roads of high grades, or on the parts of roads where grades are high. It seems to be overlooked, however, that the relative performance of heavy and light engines (comparing them now only as to their available or adhesive weight), will be nearly the same upon all grades; and hence, that if it is true economy to get all the power you can out of a locomotive on a grade of 100 feet per mile, it is no less wise to make the most of its power on a level grade. The Reading Railway, which has no ascending grade in the direction in which its coal moves (except a short one on its Port Richmond branch, at Shutadelphia, where helping power is used), employs heavy connected engines altogether in its coal trade, and by the enormous trains drawn by these engines the cost of motive power is reduced to its minimum. The truck engine does not, it should be remarked, show as unfavorably in its power of draught alongside

of the connected engine, in consequence of the free use of sand to promote adhesion; but this, it is plain, must increase the resistance to the movement of both the engine and train, and the wear of both wheels and rails, largely.

"My reasons for preferring the eight-wheel connected engine are thus stated, with, I hope, sufficient explicitness."

XVII.

THE SCOUR OF THE MISSISSIPPI RIVER.

THE bed of a river composed of material easily moved by the water, such as sand and gravel, is liable to change of form from time to time which it is important to regard in putting down foundations. Where the bottom scours badly, wooden piles are liable to be washed out; and even a rip-rap protection, by decreasing the water-way, will often bring on a scour, and be the means of undermining itself. A scour will often occur upon one side of a pier, and not on the other; thus throwing a heavy side strain upon the masonry. This movement of the bottom is especially notable in the Mississippi. The original area of water-way at the Quincy Bridge was 75.636 square feet; the area of the obstruction introduced for the piers was 10.500 feet; the area scoured out was 15.548 feet; leaving the new area 80.684 feet, or 5048 feet greater than the original area. Numerous interesting facts upon this matter are given in the exceedingly valuable reports of Captain James B. Eads, chief engineer of the magnificent bridge across the Mississippi, at St. Louis, from which the following facts are taken:—

"It is a fact well known to those who were engaged in navigating the Mississippi twelve years ago, that the cargo and engine of the steamboat America, sunk 100 miles below the mouth of the Ohio, was recovered after being submerged 20 years, during which time an island was formed over it, and a farm established upon it. Cotton-wood trees that grew upon the island attained such size that they were cut into cord wood and supplied as fuel to the passing steamers. Two floods sufficed to remove every vestige of the island, leaving the wreck of the America uncovered by sand, and 40 feet below low water mark, when, in 1856, the property was recovered. Pilots are still (1868) navigating the river who saw this wreck lying near the Arkansas shore, with her main deck scarcely below low-water mark at the time she was lost. When the wreck was recovered the main channel of the Mississippi was over it, and the hull of the vessel had been cut down by the action of the current at the bottom, nearly 40 feet below the level, at which it first rested, and the shore had receded from it by the abrasion of the stream nearly half a mile. These remarkable but well attested facts came under my own observation, and

occurred at Plumb Point, 100 miles below Cairo, where the river is more than a mile wide.

"I had occasion to examine the bottom of the Mississippi, below Cairo, during the flood of 1851, and at 65 feet below the surface I found the bed of the river for at least three feet in depth a moving mass, and so unstable that in endeavoring to find footing on it beneath the bell, my feet penetrated through it until I could feel, although standing erect, the sand rushing past my hands, driven by a current apparently as rapid as that at the surface. I could discover the sand in motion at least two feet below the surface of the bottom, and moving with a velocity diminishing in proportion to the depth at which I thrust my hands into it.

"Singularly enough, the fact is almost certain that at seasons of *lowest* water this deposit is also liable to be removed to an extent probably sufficient to lay bare the rock in mid channel. The current being much less when the water is low, the sand accumulates to its greatest depth. When the river freezes over, which only occurs when it is quite low, a strong crust of ice, from 10 to 15 inches thick, is formed at St. Louis, where the width is only 1600 feet at low water, and 2200 at high water, while there are frequently great stretches of the river above unclosed. The floating ice formed in these open spaces is carried down in large masses, which accumulate in this and in other narrow passages of the river, and form what are termed ice gorges. These accumulations sometimes extend several miles above the contracted channels of the river, and cause the water to rise or "back up" 10 and even 20 feet in some instances above its former level. The firmly frozen crust serves to hold the masses that are accumulated beneath it, and the great height attained by the "backing up" of the water above the gorge increases the currents that are sweeping below the ice to a degree probably greatly exceeding that of the floods, if we may take the water levels above the gorge as an index to the current created by this hydrostatic pressure. These currents, I believe, would prove too great to be resisted by any ordinary rip-rap usually employed to protect foundations not resting on the rock. The ice being lighter than the water, it follows that these currents will be constantly acting beneath the gorged ice, and in direct contact with the sand. As rapidly as the latter is cut away fresh supplies of ice are driven under, and thus the mass continues to grow in depth, and the current to be directed nearer to the rock. After a few weeks the pressure of the back-water becomes so enormous as to sweep the gorge away, and on such occasions the open space of water below the gorge is at once filled for miles with the submerged ice thus liberated. This ice can be readily distinguished from the crust or surface ice by its scarcely floating, and by the quantities of sand and mud with which it has been saturated during its imprisonment.

"On two occasions I undertook to cut a channel in the ice through which to remove from gorges two valuable diving-bell boats to places of safety. The undertaking was only successful in one case. The surface ice being removed from the canal and hauled off on its sides, I found the quantity of submerged ice which continually arose when that in sight was removed was so great that the supply seemed inexhaustible. In the case where I was successful I was able to cut the channel from an open part of the river up to the vessel, and through it the submerged ice was floated out, and the channel thus cleared.

"The establishment of piers in the channel of the river must facilitate the formation of an ice gorge at the bridge in the winter, and they will certainly tend to its retention until the sand is scoured about and between them to an unknown depth. For these reasons I have maintained and urged that there is no safety short of resting the piers firmly upon the rock itself."

In a subsequent report Captain Eads states that a scour of 51 feet below low water actually took place at the eastern pier of the bridge, although eminent engineers had given their opinion that 30 feet was the greatest scour to be apprehended.

XVIII.

CAISSONS FOR THE ST. LOUIS BRIDGE

REFERENCE has been made in Chapter XIV. to the method of sinking the piers for the above work. From the reports of the engineer, Captain James B. Eads, the following is extracted: —

The air-chambers of both caissons are nine feet in height, and their sides are formed of $\frac{3}{4}$ inch plate iron in the larger, and $\frac{5}{8}$ inch in the smaller one. The designs of both caissons are quite similar, except that the smaller has but five air-locks; and the plate-iron used is somewhat lighter than in the other. A description of one (which must necessarily be brief) will answer for both. The air-chamber is simply a huge diving-bell beneath the pier, being of the full size of the latter. Its roof is required to sustain the enormous weight of the entire pier, from the rock to the surface of the water, and must be of such strength as to prevent any change of form, as that would endanger the cracking of the masonry before it has been finally bedded upon the rock. As we shall have to work, in all probability, with the river not less than ten feet above low-water mark, and perhaps twenty feet, the masonry resting on the roof of the air-chamber in the large caisson will be nearly 100 feet high when the bottom of the air-chamber reaches the rock. The iron plates forming its

roof are of ½-inch thickness. Transversely over this are placed thirteen iron girders, which are securely riveted to it at intervals of five and a half feet. Each girder is five feet in height, and is made of ½-inch plate iron, with a top chord of $5 \times 7\frac{1}{2}$ inches. The spaces between them will be laid with masonry. Beneath the roof are placed two massive wooden girders, in the opposite direction to the iron ones, and these latter divide the area of each chamber into nearly three equal parts. Communication between these three divisions will be had through openings made for this purpose through these girders. These timber girders are intended to rest upon the sand, and support the roof from below, thus giving support to the iron girders at equidistant points in their length. The sides of the air-chambers are strongly braced, to resist the pressure of the sand, with plate-iron brackets stiffened with angle iron. Between the brackets, near the bottom, is placed all around the chamber a course of strong timbers, the bottom of which is level with that of the girders, and which are also designed to rest upon the sand. The area of bearing surface of this course of timber, and of the two girders in the larger caisson, is 850 square feet. The support given by this surface resting upon the sand, together with the buoyant power of the compressed air in the chamber, and the friction of the sand on the sides of the caisson, are the only means relied upon to sustain the pier in its gradual descent to the rock. Workmen and superintendents will see that the sand is evenly excavated, by which means the vertical position of the pier will be maintained. As in the diving-bell, the bottom of the air-chamber is entirely open, and the water is prevented from rising within it by the compressed air forced into it. A false bottom is placed under it, by which it is converted into a kind of boat, and in this way the caisson will be floated and towed to its place in the river.

The caisson will be held in place by 14 large guide piles, each 3½ feet in diameter, and all strongly braced together. Ten large screws, each 25 feet long, and supported by these piles, will regulate the descent of the caisson, and prevent any tilting of it until it reaches the sand. The sides of the air-chamber extend two feet below the timber girders within it, and form a cutting edge always that much in advance of the bearing surface of the wooden girders, which are 7 feet in height. This arrangement will more effectually counteract any tendency in the caisson to move horizontally in descending. When secured within the guide piles, and attached to the large screws above mentioned, air will be forced into the air-chamber, the false bottom then removed from under it, and the laying of the masonry will be immediately commenced over the roof of the air-chamber. The walls of the caisson will be extended up ten feet above the roof of the air-chamber, before being towed into position, and they will be built up from time to time by riveting on additional plates as the caisson sinks by the weight of the masonry laid within it. The weight of the masonry will be sustained by the buoyancy of the air-cham-

ber and caisson until the latter reaches the sand, the screws being only relied on to keep it steady until it enters the sand.

The large caisson will contain seven air-locks, through which to obtain entrance to and exit from the air-chamber for men and materials. The air-locks have heretofore been placed above the surface of the water, and ingress and egress to the chamber obtained through large vertical iron pipes leading through the masonry. In these caissons the air-locks are placed within the roof of the air-chamber, and access will be had to them through brick wells or air shafts, built up in the masonry over them, by which a very important saving in cost will be effected, the delay in constantly adding new joints of pipe under the air-locks as the pier descends avoided, and greater convenience attained in introducing materials and workmen into the chamber.

The air-locks are circular vertical chambers, about five feet in diameter, and of various heights — from six to twelve feet. They are made of $\frac{1}{2}$ inch plate iron. They are provided with two doors, one opening into the open air, and one into the air-chamber. The first door in opening swings into the air-lock, and the other into the air-chamber. The fact that the air-chamber is filled with compressed air involves the necessity of one or the other door being closed to prevent its escape. Either one being closed, the pressure of the air tends to keep it closed, but leaves the other free. The inner door being closed, we enter the air-lock through the outer one, and close it behind us. A cock is then opened communicating with the air-chamber, and the air-lock is immediately filled with compressed air. This equalizes the pressure on both sides of the inner door, and it can at once be opened, and we then enter the air-chamber. To return to the open air we re-enter the air-lock, close the inner door, and then open another cock, which allows the compressed air in the lock to escape. This relieves the outer door of the pressure that was on it, and it can at once be opened for our exit. One door or the other is always free to open. To open both doors at once in thirty-three feet of water would require an enormous force (about ten thousand pounds) to be applied to the one on which the air pressure was acting at the time. This gives great security to those within the air-chamber against the carelessness or ignorance of others passing in or out. The sand is removed as the caisson sinks, by means of pumps designed for the purpose. Seven sand-pumps will be used in the caisson of the eastern pier, and five in the western one for removing the sand as the piers descend. These pumps are of very simple but novel construction, never having been used before. One of three-inch bore has been thoroughly tested on the site of the bridge, in forty feet of water, and found capable of discharging ten cubic yards of sand per hour. Fifty-four cubic feet of sand were delivered by it from that depth in eleven minutes. Gravel stones as large as could enter it (two and one-fourth inches diameter) were discharged by it with as much facility as sand. As there are no working

parts in its construction, it seems scarcely possible for it to get out of order. The principle on which the pump acts is somewhat similar to that of Giffard's Injector, water being used instead of steam. A stream of water is forced down through one pipe and caused to discharge near the sand into another in an annular jet, and in an upward direction. The jet creates a vacuum below it, by which the sand is drawn into the second pipe or pump, the lower end of which is in the sand, and the force of the jet drives the sand on upwards to the surface of the river as soon as it passes through the annular opening in the jet.

The superiority of this pump over all others capable of pumping sand and gravel, with which I am acquainted, consists in its being supplied with the requisite quantity of water for keeping the sand in a fluid condition, whilst the suction-pipe is inserted directly into the sand. With other pumps the quantity of sand cannot be regulated, and it is liable to be drawn in in such quantity as to choke up the pipes and working parts of the pump, the water drawn in with the sand not being sufficient to keep it fluid. The weight of the column set in motion by the pump is consequently liable to constant variation, as it has more or less solid material in it; and when the column is raised by atmospheric pressure (vacuum), and a lift of only twelve or fifteen feet attempted, the pump may cease to lift the column, it being so much heavier than water, because of having so much sand in it. In this case, if the suction pipe has a foot-valve at the lower end of it, as most centrifugal pumps do, the sand falls back on it, and the pipe must be taken out to empty it of its contents. When too much sand is in the column above the ordinary pumps, the pipe is liable to be overloaded with it, the action of the pump ceases, and the contents must be removed before the pump will again work. If the water jet in the pump described fails to discharge the sand, no choking up of the pump can occur. There is nothing in the construction of it to prevent the sand from running back when the jet stops, and if the jet is strong enough it will raise it to any height. The one experimented with worked admirably even with the end of the pipe, 19 feet deep in the sand, and 40 feet depth of water over the sand. The vacuum obtained by the jet with the pump out of water, and the lower end closed, supported $28\frac{3}{4}$ inches of mercury, the vacuum being almost perfect.

The interruption and annoyance caused by the necessity of riveting on the iron plates outside of the masonry, to exclude the water as the pier descended, led to modifying the design for the later caisson, by which the use of these plates could be abandoned after the first 29 feet; 9 feet of this height being the air-chamber, and the remaining 20 feet enveloping the masonry above the chamber. This height of plate iron was deemed requisite to give such rigidity to the caisson as would insure it against any twisting or straining that would endanger the bond of the masonry. After a depth of 40 or 50

feet was reached by the east pier, it was found that brick linings in the shafts, although surrounded by many feet of masonry carefully laid in hydraulic cement were not sufficient to exclude the water which at this depth filtered through quite rapidly. To prevent this, and enable the iron on the outside of the pier to be dispensed with, on the west pier its shafts were lined with white pine staves, 3 inches thick in the centre shaft, which was 10 feet in diameter, and $2\frac{1}{2}$ inches thick in the smaller wells, which were 4 feet 9 inches in diameter. This device answered admirably, and the estimated saving in plate iron over the original design was about $10,000. The lower staves were, however, found to be too weak to sustain the high water of the spring freshet without expansion bands of 1×3 inch bar iron, which were placed against them in the shafts. This novel feature of wooden linings and no exterior envelope for the masonry was adopted for the caisson for the east abutment pier, the staves in the main well, which is 10 feet in diameter, being 10 inches thick in the lower part, gradually diminishing to 3 inches at the top.

The air-chamber of the east pier of the work above referred to was upon completion filled solid with concrete. The pier thus rests upon a large bed of concrete. At the east abutment pier, however, a wall of concrete, of an average width of $3\frac{1}{2}$ feet, and a depth of 2 feet 6 inches, was laid beneath the timber sides of the caisson, and the interior area was filled with sand, which, thus confined, makes as solid a foundation as the rock itself.

XIX.

NARROW GAUGE RAILWAYS.

"EVERY inch added to the width of a gauge beyond what is absolutely necessary for the traffic adds to the cost of construction, increases the proportion of dead weight, increases the cost of working, and in consequence increases the tariffs to the public, and by so much reduces the useful effect of the railway.

"I shall proceed to show that if its gauge (the London and North Western Railway) were 3 feet, instead of 4 feet $8\frac{1}{2}$ inches, its goods traffic could be hauled at half the present cost, with half the present motive power, and in such a way as to reduce the present tonnage over the road by a half." — *Paper read before the British Association, at Liverpool, in* 1870, *by Robert F. Fairlie.*

"Where cheap roads are practicable, the use of the narrow gauge may reduce the cost about one half, without reducing the necessary efficiency.

"The cost of hauling a ton of goods over a narrow gauge road may be safely reckoned at less than one half the cost of hauling the same goods over a road of common gauge." — *Report of Committee of Massachusetts Legislature on Narrow Gauge Railroads,* 1871.

"A first-class railroad cannot be constructed and operated with a gauge narrower than 4 feet $8\frac{1}{2}$ inches, that will, if doing a large and miscellaneous business, combine equal speed, comfort to passengers, and capacity for freight, with as much facility and economy as the same elements can be combined upon the 4 feet $8\frac{1}{2}$ inches, or even a broader gauge." — *Silas Seymour*, 1871.

"Another fallacy upheld by the advocates of very narrow gauge lines is, that the narrower the gauge the less — almost in an equal degree — is the proportion of dead weight to paying load; whereas, both theory and experience go to prove that so long as the amount of accommodation and the strength to resist hauling and buffing strains are constant, the proportion of dead weight to paying load is almost, if not entirely, independent of the gauge." — *Van Nostrand's Magazine*, Vol. IV., p. 245.

The above extracts will serve to show the difference of opinion among engineers in regard to a subject which has attracted much attention of late. The new "Battle of the Gauges" bids fair to be fought with as much vigor as that which marked the days of Stephenson and Brunel; and if it has no other result, it will certainly call attention to some of the prominent defects of the present system of transportation upon railways. It is claimed for the new system of narrow gauge roads, —

First. That less capital is required for the railway.
Second. That less capital is required for the rolling stock.
Third. That steeper grades and sharper curves are admissible.
Fourth. That the proportion of non-paying load is reduced.
Fifth. That the capacity is equal to any ordinary traffic.
Sixth. That even a break of gauge between the new system and the common width, involving the transshipment of freight, is less expensive than the extra cost of transport over the ordinary (4 feet $8\frac{1}{2}$ inch) gauge.

That less capital is required for the construction of a narrow gauge road no one will deny; the point upon which engineers differ is — how much less. The advocates of the new system claim a saving in this respect all the way from one to two thirds. The opponents deny that a saving of more than from 5 to 10 per cent. can be made. It is claimed by one party that the cost of construction is directly proportional to the width of gauge. By the other party it is maintained that the only saving is in the small vertical section taken out in reducing the width.

Mr. Pihl, a distinguished Norwegian engineer, who has introduced a gauge of 3 feet 6 inches into Norway and Sweden, gives the following as the comparative cost per mile of railways of different gauges: —

In the amounts in the table on next page, the value of the pound sterling is reckoned at $5.00.

Width of gauge.	4 ft. 8½ in.	3 ft. 6 in.	3 feet.	2 ft. 9 in.	2 ft. 6 in.
Formation width.	15 ft.	13 ft. 6 in.	13 feet.	12 ft. 6 in.	12 feet.
Excavation.	$3.109	$2.909	$2.843	$2.776	$2.710
Drainage works.	365	350	345	340	335
Bridges.	2.025	1.915	1.875	1.837	1.800
Ballast.	2.010	1.570	1.506	1.474	1.442
Sleepers.	1.088	855	776	738	699
Iron, 45 pounds rail.	3.134	3.134	3.134	3.134	3.134
Fencing.	750	750	750	750	750
Stations.	675	675	675	675	675
Contingencies 20 per ct.	2.631	2.432	2.381	2.345	2.309
Total.	$15.787	$14.590	$14.285	$14.069	$13.854

In 1871 the Massachusetts Legislature appointed a committee to examine the merits of the narrow gauge system, from the report of which we have the following comparative estimates for a 4 feet 8½ inch and a 2 feet 9 inch gauge, the average depth of cutting and embanking being taken at 4 feet : —

ITEMS.	Gauge 2 Feet 9 Inches.	Gauge 4 Feet 8½ Inches.
Rails.	$4.243	$6,600
Sleepers.	352	924
Spikes.	175	264
Joint fastenings.	400	700
Laying track.	250	325
Embankment.	1.513	2,151
Cuttings.	1.480	1.927
Rock cutting.	1,611	2.085
Ballast.	1 000	2.000
Sidings.	200	334
Masonry and bridges.	1.140	2.000
Total cost of road per mile.	$12.364	$19.310

Between the two estimates above is the very considerable difference shown below: —

ESTIMATES.	2 Feet 9 inch Gauge.	4 Feet $8\frac{1}{2}$ Inch Gauge.	Difference.	Per cent. of saving.
Mr. Pihl's estimate, . .	\$14.069	15.787	1.718	10.9
Massachusetts estimate,	12.364	19.310	6.946	36.0

If we make the narrow gauge ties 5 feet long, the ballast 7 feet wide on top and 18 inches deep, the ballast slopes 1 to 1, and the ditches 1 foot wide, we have the width at sub-grade 12 feet, or 6 inches less than Mr. Pihl gives it. If we make the 4 feet $8\frac{1}{2}$ inch ties $8\frac{1}{2}$ feet long, the ballast 12 feet on top and 2 feet deep, and ditches 1 foot, the sub-grade width is 18 feet. With these dimensions the area of the section for a depth of 4 feet is 72 square feet for a 2 feet 9 inch gauge, and 96 feet for the 4 feet $8\frac{1}{2}$ inch gauge, or a saving of 24 feet of area, or just one fourth of the amount of excavation. The greater the depth of cutting, the less this percentage of saving. Thus, for an 8 feet cut the saving is 20 per cent.; for a 16 feet cut, $14\frac{1}{3}$ per cent.; and for a cutting 24 feet deep, 11.1 per cent., or less than half that for a 4 feet cutting. In the Massachusetts estimate the saving in excavation is 23.2 per cent.; in Mr. Pihl's estimate the saving in excavation for the same reduction in gauge is only 10.7 per cent. For shallow work the Massachusetts estimate would be the more correct; for deep work Mr. Pihl's estimate would be nearer the truth. Mr. Pihl's estimate for iron is the same for all gauges; which is, of course, incorrect. The Massachusetts estimate reduces the amount of ballast by 50 per cent.; Mr. Pihl by 26 per cent. The Massachusetts estimate reduces the cost of sleepers from \$924 to \$352: Mr. Pihl from \$1088 to \$738 only. The Massachusetts estimate reduces the cost of bridging and masonry from \$2020 to \$1140, or 43 per cent.; Mr. Pihl reduces bridges from \$2025 to \$1837, or 9 per cent. only. As far as bridge masonry and culverts go, the percentage of saving will, of course, be more with light work and less with heavy work. Bridge trusses for the same span will be considerably less in depth and width, and much lighter when the span is not large and the weight of the bridge itself is only a small part of the whole. Probably for an average country a mean between the percentages of saving by the two estimates above would not be far from the truth, or say 25 per cent.

In considering the first cost of making the road-bed there is another point to be regarded. It has been shown above that the lighter the excavation and embankment the larger the percentage of saving by the reduction of gauge. One strong claim of the narrow gauge advocates is, that it permits the use of sharper curves and steeper grades than are feasible with the common width; thus

enabling us to avail ourselves of the large percentage of saving allowed by the lighter excavation and embankment. The least radius of curvature upon the famous 1 foot 11½ inch gauge road in Wales, from Festiniog to Port Madoc, is 116 feet (1¾ chains), in lengths of from 80 to 200 feet, and the steepest grade 87 feet per mile. Upon the Broelthal line, near Cologne, in Germany, gauge 2 feet 7 inches, there are curves of 125 feet radius (38 metres), and grades of 66 feet per mile. "The use of the narrow gauge," says the Massachusetts report, "permits the employment of curves entirely impracticable for roads of common width; the minima of the two gauges (4 feet 8½ inches and 2 feet 9 inches) being practically as one to three, or, say 220 feet and 660 feet, respectively, and it considerably increases the grade maximum as well."

Upon this point we believe an error has been introduced into the question. The difference in length between the inner and outer rails, and thus the amount by which the wheels must slide, of course increases with the gauge; but the resistance from curvature depends very much upon the *length* of the train. For the same number of square feet of floor surface of cars, if we reduce the width, we must of course increase the length. When we consider the effect of curvature upon a given capacity of train, we shall find that the narrow gauge does not have the superiority claimed for it. A four-wheeled engine with 3 feet wheels placed 6 feet apart from centre to centre, will traverse curves upon a 4 feet 8½ inch road as sharp as any of those yet built upon the narrow gauges. With regard to grades, the effect of gravity is so much per ton for a given incline, no matter what the gauge. In estimating the resistance to motion, it is also to be remembered that the large wheels of the 4 feet 8½ inch cars will run easier than the smaller wheels of the narrow gauge. But whatever dead weight is saved, releases just so much of the locomotive power; and this released power may be applied to overcome extra resistance upon sharper curves and steeper grades.

With regard to the saving in first cost of the equipment, we have the following: The passenger cars upon the 4 feet 8½ inch and the 3 feet 6 inch gauges of Norway, are stated by Mr. Pihl, to cost $1640 and $1150 respectively. The freight cars, $450 and $320. The estimate in the Massachusetts report, before referred to, for a road 25 miles long, is as follows:—

Gauge 4 feet 8½ inches.

3 engines,	$48,000
5 passenger cars,	20,000
2 mail and baggage cars,	2,600
30 freight cars,	30,000
20 freight cars,	16,000
	116,600
Or, per mile,	4,664

Gauge 2 feet 9 inches.

3 engines, ...	$18,000
5 passenger cars, ...	5,500
2 mail and baggage cars, ...	1.200
35 freight cars, ...	6,125
25 freight cars, ...	4.000
	$34,825
Or, per mile, ...	1,393

We are, in the Massachusetts estimate, given to understand that the cost per mile for equipment upon the 4 feet 8½ inch and the 2 feet 9 inch gauges will be as 4664 to 1393; but the relative power and capacity of the two equipments above are disregarded. If these were taken into account, we should find the relative cost of equipment for the same accommodation upon the two gauges to be very different from that above stated. The 4 feet 8½ inch gauge passenger car above is estimated at $4000, the 2 feet 9 inch car at $1100. If we reckon the narrow gauge car to be 25 × 6 feet, and the 4 feet 8½ inch car to be 9 × 45, the relative capacity and cost would be —

WIDTH OF GAUGE.	Cost.	Square Feet.	Cost per Square Foot.
4′ 8½″ gauge, ...	$4.000	405	9.88
2′ 9″ gauge, ...	1,100	150	7.33

In like manner the engines, costing in one case $48,000, and in the other $18.000 only, must be reduced to a common standard of efficiency, when the proportion will be very different from that given in the Massachusetts report. Correcting the above statement of the total cost per mile for equipment, and putting in its place the cost of equipment per unit of accommodation furnished, we shall find the 4 feet 8½ inch gauge to exceed that of the 2 feet 9 inch gauge by perhaps 50 per cent., instead of by over 200 per cent., as given in the Massachusetts report.

Upon no point do the advocates of the narrow gauges rely more than upon a large saving of dead weight in the rolling stock, and upon no point are they more vigorously combated by the opponents of the system.

"It is known, and everywhere admitted," says Mr. Fairlie, "that the proportion of non-paying to paying weight in passenger trains is as much as 29 to 1; and in goods trains, exclusive of minerals, as much as 7 to 1. This terrible disproportion is partly due to the system of management pursued, but in a far greater degree to the gauge. The dead weight of trains, conveying either

passengers or goods, is in direct proportion to the gauge on which they run; or, in other words, the proportion of non-paying to paying weight (as far as this is independent of management), is increased exactly as the rails are farther apart; because a ton of materials disposed upon a narrow gauge is stronger, as regards its carrying power, than the same weight when spread over a wider basis. In proof of this proposition, I need only cite the case of the Festiniog Railway, with its gauge of one foot 11½ inches. The wagons used upon it for carrying timber weigh only 12 hundred weight, and they frequently carry a load of over 3½ tons, and at a speed of 12 miles an hour. In other words, these wagons carry as much as six times their own weight, whilst the best wagons on the ordinary English narrow gauge do not carry as much as twice their own weight."

As opposed to this statement, we have the following from a leading English engineering journal: "In the case of the extremely narrow Festiniog Railway, we find the dead weight of the passenger stock to be greater per passenger carried than many carriages running on the Continent on 4 feet 8½ inch gauge lines, these latter carriages, notwithstanding, affording a greater number of cubic feet of capacity per passenger, and being mounted on 40 inch in place of 18 inch wheels. Similar evidence might be adduced concerning the goods carrying stock, were it necessary to do so; but we need only refer to the carriages and wagons in general use on ordinary lines 15 or 18 years ago to prove that light stock can be made for the 4 feet 8½ inch, or a wider gauge, if the general nature of the traffic warrants its use."

The figures above given by Mr. Fairlie are largely in excess of those obtained from railways in the United States. To carry one ton of passengers upon the roads of New York requires 14 tons of equipment, and to carry a ton of freight only 1.4 tons of equipment; while in Massachusetts the transport of a ton of passengers involves the moving of only 6 tons of equipment, and a ton of freight 2.3 tons of equipment. No conclusion of any value whatever can be drawn by comparing the load which a narrow gauge car *can* carry with the load which the wide gauge cars *do* carry in common practice; nor are the proportions of paying to non-paying load, deduced from small roads hauling coal, slates, and other minerals, and running always well filled, to be compared with the proportions deduced from the ordinary conditions of a miscellaneous freight traffic. The conditions under which the two gauges work must be the same, and then we shall find a very moderate saving in dead weight by the narrow gauge, compared to that claimed by its advocates.

Thus Mr. Pihl reduces the dead weight per passenger only from 0.20 to 0.17 of a ton from the 4 feet 8½ inch to the 3 feet six inch gauge, and for freight, from 0.69 to 0.55 of a ton of dead weight per ton of load. The weight of the wagons on the little Bröelthal Railway, gauge 2 feet 7 inches, is 2½ tons, and

the load carried 5 tons. The common eight-wheeled platform car weighs 16,000 pounds, the box car 20,000 pounds, and these cars will carry from once and a half to twice their own weight. In widening a car we may suppose about half its weight to be increased simply in proportion to the width, but the other half will be increased more, on the ground that the strength of a beam placed horizontally is inversely as its length: if we double the weight of a car, the transverse framing is only half as strong, unless it is made deeper. The wagons upon the Festiniog Railway, Mr. Fairlie states, carry as much as six times their own weight, whilst the best wagons upon the ordinary English narrow gauge do not carry as much as twice their own weight. The wagons upon the Festiniog road carry six times their own weight because they run well filled with a compact mineral: carrying ordinary mixed freight, they would do no such thing. An eight-wheeled car, says the Massachusetts report, of the 4 feet $8\frac{1}{2}$ inch gauge, weighs nearly 10 tons, and carries a burden of 10 tons, while a four-wheeled car of the 2 feet 9 inch gauge weighs 3 tons, and carries a burden of 6 tons. If the box car is intended in the above remark, which is the only car on the 4 feet $8\frac{1}{2}$ inch gauge that weighs 20,000 pounds, and carries the same, its capacity is $24 \times 8 \times 8$, or 1536 cubic feet. If the four-wheeled 2 feet 9 inch gauge box car is 12 feet long, 5 feet wide, and 6 feet high, its capacity is 360 cubic feet. The more correct mode of comparing the box cars, which are used chiefly for the lighter but more bulky freight, is by cubic capacity, and not by the maximum weight they will carry. This comparison is as follows: —

$4'\ 8\frac{1}{2}''$ gauge, cubic capacity per ton of dead weight $\frac{1536}{10}$, or 153.6 feet.

$2'\ 9''$ gauge, cubic capacity per ton of dead weight $\frac{360}{3}$, or 120.0 feet.

The 4 feet $8\frac{1}{2}$ inch eight-wheeled flat car weighs 16,000 pounds, and carries 24,000 pounds, or, if we call it 24×8 feet on the floor, 125 pounds per square foot. If we put 140 pounds per square foot on the 2 feet 9 inch car, and call the floor 5×12, the whole load is 8400 pounds, and we need three cars to carry the 24,000 pounds carried by the one 4 feet $8\frac{1}{2}$ inch flat car; and for the dead weight to be the same in both cases, the narrow cars should not weigh over 5333 pounds each. Therefore, notwithstanding the statements made as to the saving in dead weight by the use of the narrow cars, and however economically compact freight, such as minerals, may be hauled, there is no reason to suppose that any very great gain can be made in the transport of the merchandise carried upon our railways.

It has been claimed that small cars will be more likely to be well filled than wide ones; that way stations often require the use of a part only of a large car, the remainder being left idle at the station. There may be some force in the latter part of the above claim, but as a general thing we have no more

right to assume that the narrow gauge cars will always run full than that the wide ones will. If there is any advantage in running a large number of light trains instead of a small number of heavy ones, we can avail ourselves of such benefit as well upon a wide as upon a narrow gauge. But the whole tendency of railway practice, especially as regards freight, is to increase the size and to decrease the number of trains.

Mr. Pihl gives the following comparative weights and capacities for the 4 feet $8\frac{1}{2}$ inch and the 3 feet 6 inch gauge cars, as actually made in Norway: —

GAUGES.	Length in Feet.	Width in Feet.	Weight of Car — Tons.	Capacity.
Passenger car, 4′ $8\frac{1}{2}$″ . .	20	7.62	6.40	32 passengers.
" " 3′ 6″ . .	20	6.83	5.45	32 "
Freight car, 4′ $8\frac{1}{2}$″ . .	16	7.33	4.15	6 tons.
" " 3′ 6″ . .	16	6.50	3.30	6 "

From these figures, we deduce the following, showing the relative capacity and weight of wagons upon the two gauges: —

ITEMS.	4′ $8\frac{1}{2}$″ Gauge.	3′ 6″ Gauge.
Square feet allowed per passenger, ...	4.76	4.27
Weight of car per passenger (tons), ...	0.20	0.17
Weight of freight car per ton of load, ...	0.69	0.55
Ratio of dead to gross load in freight car, ...	0.41	0.35

These figures do not show the large saving in dead weight so strongly claimed by the narrow gauge advocates; for even allowing the above narrow gauge freight car to carry as much as the wide car, viz., 6 tons, the difference in the ratio of the dead to the gross load is only the difference between 41 and 35 per cent. So, too, in regard to the passenger cars, putting the same number of passengers into both, the dead weight is only .03 of a ton less per passenger in the narrow than in the wide car. The above cars are in both cases those in actual use upon the Norwegian railways. With regard to the locomotive power, for the same total load hauled there must be the same weight on driving wheels, whether the load be in one train or a dozen. So far as the dead weight of cars is reduced, so far also may the weight of engines be

reduced, provided we do not need extra adhesion on account of steeper grades and sharper curves. Engines may be made, and have been made, for small roads on a 4 feet 8½ inch gauge that work with very great economy, and that will traverse curves as sharp as any upon the narrow gauge roads. The little Rocky River Railroad, a line six miles long, from Cleveland to Rockport, Ohio, gauge 4 feet 8½ inches, is worked by a tank engine, by the Baldwin Locomotive Works, weighing 16,000 pounds, having cylinders 7 × 14, and one pair of 42-inch driving wheels. It is guaranteed to haul 235 gross tons on a level. With 125 pounds of steam it has taken six cars, containing upwards of 400 passengers, from one end of the road to the other in 13 minutes, making one stop, the steepest grade being 53 feet per mile.

With regard to the *capacity* of narrow gauge roads, the statements commonly made involve an error. Because in certain places narrow gauge cars have transported a comparatively large tonnage of minerals upon a short road, it is assumed that they would carry a corresponding tonnage of the miscellaneous freight generally offered upon long roads. Thus, in the Massachusetts report, we are told that the tonnage on the Festiniog road in one year was 136,132, or 9,388 tons per mile of road, or "more than double the traffic per mile of the Connecticut River Railroad." This statement is of no value whatever, since, if the Festiniog slate quarries were 100 miles from Port Madoc, instead of a dozen, the traffic per mile would be correspondingly reduced. The ultimate capacity of the narrow gauge roads has been based upon the fallacy that an exceptional traffic affords a rule for estimating the capacity for the ordinary kinds of business. We apprehend that but one opinion would be given by the managers of such a line as the Pennsylvania Railroad as to the economy or even the possibility of working their immense freight traffic upon a gauge of 2½ or 3 feet.

Perhaps the fairest exposition of the fallacy that infests the narrow gauge advocates with regard to the capacity of their roads, certainly the most authoritative expression of opinion in regard to this matter, is that given by Mr. Fairlie in the paper read before the British Association, at Liverpool, in 1870, from which the following is extracted: —

"On the 4 feet 8½ inch gauge, the proportion of non-paying to paying load has been taken at 4 to 1, although it has proved largely in excess of this. The wagons employed average four tons in weight, so that on this reckoning each wagon carries one ton for every mile it runs. The wagons for a line of 3 feet gauge weigh each one ton, and carry a maximum load of three tons. Supposing that the same number of wagons and trains were run on the narrow gauge as on the broad, it follows that the average one ton of merchandise now carried would easily be taken in a wagon weighing one ton instead of four tons, and that the gross load passing over the line for one year would be only 20,000,000 of tons, instead of 50,000,000; while the same amount of pay-

ing weight would be carried in either case. That is, the small wagons which are capable of carrying three times the weight of goods now actually carried in a four ton wagon, would only have to carry one third of that quantity, and would produce the same paying load as the heavier wagons, and as the haulage cost is precisely the same whether the tons hauled consist of paying or non-paying load, it follows that this expense would be reduced to two fifths of what it now is. If the same number of trains were to run per day, the weight of each would be reduced from 255 tons to 102 tons; or, if the same gross weight of train was employed, the number of trains per day would be reduced from 626 to 250. If there should be sufficient traffic to load the narrow gauge wagons in such a way as to require the same number and weight of trains that are now worked, the result would be that, without increasing by one penny the cost of haulage and of the permanent way expenses, the 3 feet gauge would carry a paying load of 25,000,000 tons as against the 10,000,000 now carried. Here then we have established the fact that, as far as capacity goes, the narrow gauge is superior to the broad one. The former can produce 25,000,000 net out of a gross tonnage of 50,000,000; whilst the latter, to produce the same result, if continued to be worked as it now is, would require that 125,000,000 tons should be hauled, and that at an increased cost in the same proportion of 125,000,000 to 50,000,000."

Suppose we have a ton of pig iron to be moved from one point to another, 100 feet distant. We put it on to a small truck propelled by an Irishman, the man and car weighing together 224 pounds. Result — paying load, 2240; dead weight, 224; ratio of non-paying to paying weight, 1 to 10. Certainly a very good show. Suppose again, that we have a ton of furniture to be picked up at ten different places, a mile apart, and that to haul it we employ a two horse team, weighing altogether one ton. Result — average paying load half a ton, dead weight one ton; ratio of non-paying to paying weight, 2 to 1. Certainly a result much inferior to that obtained with the pig iron. Now, according to Mr. Fairlie, all we have to do to improve this latter state of affairs is to send off our Irishman with his truck, which we know will carry ten times its own weight, to get this ton of furniture. The question is whether we will send a ton of equipment for the furniture, or 224 pounds. Shall we reduce the ratio of non-paying to paying load from two to one tenth? Absurd as this reasoning seems, it is nevertheless a fair specimen of the great dead weight argument of the advocates of narrow gauges. Upon just such logic as this depends the conclusion that the capacity of the narrow gauge is equal, or, as Mr. Fairlie says, superior, to that of the wide gauge. It is not a question of square or cubic feet of carrying capacity, but a commercial question, governed by the movements of articles of trade.

The cost of doing work upon a railway depends both upon the amount of interest to be paid upon the capital, and upon the actual cost of maintaining and operating the road. So far as the cost of construction is reduced, the amount of interest, and thus one part of the cost of transport, is reduced also.

The interest being independent of the amount of traffic, this reduction is less per ton as the number of tons carried becomes greater. Upon a short road doing a large business it would be inconsiderable. Upon a long road doing a small business it would be important. Captain Tyler, government inspector of railways in Great Britain, concludes that a system of narrow gauge lines could be built costing two thirds of those now constructed, and maintained for three fourths the expense. Reduction of dead weight would of course reduce the cost of repairs of engines, cars, and track, and would also save fuel, oil, waste, etc. But the reduction of dead weight is very small compared with what has been claimed. Probably, regarding the bulk as well as the weight of freight that could be transported upon the different gauges, a saving of 20 per cent. of the dead weight is all that could be effected by a reduction from 4 feet $8\frac{1}{2}$ inches to 3 feet. If we reckon half of the whole expense of operation to be in proportion to the gross weight hauled, and the ratio of dead to gross load to be two thirds (which is an average of that obtained in Massachusetts and New York),

$$\text{we should save } \frac{20}{100} \text{ of } \frac{66}{100} \text{ of } \frac{50}{100}, \text{ or 6.6 per cent.}$$

of the working expenses. If we have a road 25 miles long, and reckon the first cost to be reduced from \$30,000 to \$20,000 per mile, and if the cost of maintenance and operation for the ordinary gauge is taken at \$5000 a mile, or \$125,000 in all, and if we carry annually 100,000 tons, the comparative cost per ton transported would be as below: —

Ordinary Gauge.

$$\left(\frac{6}{100} \text{ of } 750{,}000 + 125{,}000\right) \div 100{,}000, \text{ or 170 cents.}$$

Narrow Gauge.

$$\left(\frac{6}{100} \text{ of } 500{,}000 + 116{,}750\right) \div 100{,}000, \text{ or 146 cents.}$$

The saving is thus 14 per cent. in place of the 50 per cent. claimed by Mr. Fairlie, and also claimed in the report of the Massachusetts legislative committee.

A prominent objection to the narrow gauge has been the break that would occur at the connection with lines upon the wider gauge. This involves the transshipment of freight, and thus introduces expense and delay. The mere cost of unloading and loading is not great, ranging from 7 to 10 cents per ton. In regard to certain classes of freight the narrow gauge car bodies might be transferred from their own wheels to the platform of the wide cars; and although this would involve the hauling of extra dead weight, it might be

cheaper than to break the bulk; grain in bulk, coal, iron ores, etc., need not be handled, but may be dumped or dropped from suitable wagons, as in the case of the coal cars in common use. The mere cost of transshipment is not, however, the whole expense. Delays would necessarily occur, since the accommodation required for the narrow gauge freight might not always be found upon the wide cars. The cost of transshipment would depend upon the character of the material to be moved; while the expense of transshipment per ton per mile depends upon the distance over which the freight is carried. For long distances it would be very little. If a narrow gauge road 25 miles long costs $10,000 less per mile for construction and equipment than a wider gauge road, and carries 100 tons of freight a day, or, say 30,000 tons a year, the saving is 50 cents per ton carried, and if transshipment costs 10 cents per ton, we save 40 cents per ton, or $12,000 a year. With 150,000 tons a year there would be no saving, the cost of transshipment just balancing the $15,000 of interest saved.

If narrow gauge roads are to be introduced as a system, connecting with each other, they should, of course, have a common width. For separate branches it might not matter whether the gauges were alike or not, though as uniformity in such cases would cost little or nothing it would certainly be advisable to establish a standard at the outset. The manufacture of the rolling stock would thus be effected with the greatest economy. The Massachusetts report recommends the legalizing a gauge of 3 feet for narrow lines, and prohibiting the use of any other. In Canada, Norway, and Russia, 3 feet 6 inches has been adopted. Gauges of 2 feet 6 inches are common in mining districts. The Denver and Rio Grande Railroad, from Denver city to El Paso, 850 miles, is to be 2 feet 9 inches. A majority of a committee of engineers appointed by the secretary of state for India, recommended 2 feet 9 inches for that country. For the Texas Pacific Railroad, 1500 miles, General Buell advises 3 feet 6 inches. For general adoption, probably 3 feet would combine as many advantages, regarding both cost and capacity, as any width that could be selected.

Briefly to review what has preceded, we may conclude as follows, regarding the several claims made for the narrow gauge railways: —

The amount of construction capital may be reduced by reducing the gauge as much as 30 per cent. in light work, and 10 per cent. in heavy work. The cost of equipment may be reduced from 25 to 50 per cent., the amount of accommodation regarding both weight and bulk of freight remaining the same.

Steeper grades and sharper curves than are admissable upon a wide gauge may be applied to the narrow roads; for, while the effect of gravity and the whole resistance upon curves will be practically the same for a given load, the dead weight saved in the narrow train releases just so much of the locomotive power, which may be applied to overcoming extra resistance upon sharper

curves and steeper grades. In this case, however, the total weight of the locomotive will be the same upon both gauges.

The proportion of non-paying to paying load is reduced; but not to anything like the extent that has been claimed. The supposed saving in dead weight by reduction of gauge has been obtained by comparing the load a narrow gauge car *can* carry, under exceptional conditions, with the load the wide cars *do* carry under ordinary conditions Probably under favorable circumstances 20 per cent. of the dead weight may be saved.

The capacity of the narrow gauge roads has been based upon the fallacy that an exceptional traffic affords a rule for estimating the capacity for the ordinary kinds of business. Because certain small, narrow gauge cars have hauled a large weight of compact mineral, it is assumed that the ratio of dead to gross load would be equally favorable in transporting the mixed freight commonly offered upon railways, much of it of great bulk but of little weight. Under the same conditions of traffic the capacity of narrow gauge roads is inferior to that of the wider roads. The general experience of railway managers, both in Europe and America, has been in favor of increasing the weight and decreasing the number of trains. Upon the narrow gauge roads precisely the reverse must be done. Just what capacity the narrow gauge roads would have for the ordinary miscellaneous freight cannot be stated. It is doubtless ample for branch lines, and for main lines doing a moderate business, but not sufficient for a first class trunk line.

The break of gauge, for the amount of traffic which a branch or a line of small traffic would have, is not objectionable; since the saving in interest on first cost more than balances the cost of transshipment. Upon a first-class road, or where the business is large, the cost of transshipment will exceed the saving by interest, and the break of gauge will be objectionable.

If narrow gauges are to be introduced, economy will be secured by the establishment of some one standard to which all lines shall conform. For this standard the width recommended by the committee of the Massachusetts legislature, viz., 3 feet, will probably combine as many advantages as any other.

XX.

THE RELATION OF RAILWAYS TO THE STATE.

THE following is extracted from Lardner's Railway Economy; and though written many years ago, is the best statement of a question which is growing more important every year: —

Railways, when first brought into operation, were regarded as exceptional

modes of conveyance, suitable to particular localities and particular conditions of commerce and intercourse. As their powers were gradually developed, it became evident that they were destined to play a more important part in the business of transport, and that they must ultimately become the general, if not the only means by which the internal movement and commerce of peoples, and even the intercommunication of people and people, would be conducted. Under this point of view, the question of their relation with the state became one of capital importance.

Hitherto the public highways in all countries have been regarded as within the special domain of government. By government and by the legislature they were controlled and regulated; and it was natural, therefore, to conclude that the same system of regulation and control must be extended to the new ways of communication, by which they seem destined to be superseded.

Between the common high roads, however, and the railways, an important difference was not slow to unfold itself. The superintendence and control of the state over the highways had been limited to their maintenance and superintendence, and to the regulation of their police. The carrying business conducted upon them was always in the hands of the public, and was regulated and controlled by the wholesome influence of competition.

The operation of the same principle of competition was contemplated in the infancy of railways, as is apparent from the provisions in the legislative enactments, by which the companies have been incorporated. It was expected that the public should be admitted to exercise the business of carriers upon them, subject to certain specified regulations and by-laws.

It soon became apparent, however, that this new means of transport was attended with qualities which must exclude every indiscriminate exercise of the carrying business. A railway, like a vast machine, the wheels of which are all connected with each other, and whose movement requires a certain harmony, cannot be worked by a number of independent agents. Such a system would speedily be attended with self-destruction. The organization of a railway requires unity of direction and harmony of movement, which can only be attained by the combination of the entire carrying business with the general administration of the road. Hence it followed, as a necessary consequence, as has been already explained, that the companies originally established for the construction of a road only, became, in spite of themselves, the exclusive carriers upon it; and hence arose, inevitably, as many local monopolies of transport as there were separate and independent companies.

This evil was speedily aggravated by amalgamation. The very same principles and conditions which rendered it indispensable that each company should have the sole direction and management of the entire movement of transport upon its own line, rendered it scarcely less expedient that systems

of lines running into each other should either voluntarily establish a code of regulations to secure their mutual harmony, or that they should coalesce so as to form fewer companies of greater magnitude. Both of these expedients have been resorted to. Lesser, placed near greater companies, have coalesced with them. A great number of small monopolies have, by the operation of the affinities of commercial interest, been drawn together, and have become a small number of great monopolies; and so indispensable has a certain unity of management and harmony of movement proved to be to the efficiency of the entire system, that, where amalgamation has not been effected, the device of the clearing-house has been invented to surmount, as far as is practicable, those difficulties which might arise from the absence of unity of direction and management of inter-communicating lines.

Such were the circumstances out of which sprung those colossal monopolies among which the territory of the United Kingdom is parcelled out, and by which the entire internal commerce, and correspondence, and personal intercourse of its people are conducted.

A great variety of relations have arisen out of a like state of things in other countries, according to the local circumstances attending the form of government, and the social and commercial condition of the people.

In some the state has taken upon itself the entire charge of the construction and working of the railways. This is the case, for example, in Belgium and Hanover, in some of the Northern Duchies, in the Grand Duchy of Baden, in Wurtemburg, Bavaria, and Austria. It is true that a few isolated lines in these several states had been conceded to companies before the great question of the relation between the state and the railways had been raised; but these cases, besides being exceptional, have gradually been diminished in number by the governments respectively redeeming the property in the roads.

In other countries, a mixed system has been pursued. Some railways have been constructed and furnished by the state, but farmed by companies on terminable and frequently short leases; the state maintaining a certain control, regulated by the clauses of the leases. In some cases the railways have been constructed and stocked by the companies themselves, who hold the property under a lease of more or less extended duration; but still the state is represented in the administration of the railway by the presence of an agent, who is invested with almost unlimited control over the working of the lines. In France, this agent was established under the name of a Royal Commissioner, and one such funtionary was nominated to form part of the administration of each railway company. Besides this, the government appoints the police of the road, all these functionaries, however, of every grade, being paid by the company. On the expiration of the leases, the state is usually bound to reimburse to the company the estimated value of

the movable stock attached to the establishment; and the company, on the other hand, is bound to sustain this movable stock in a satisfactory and efficient state pending the lease.

In cases where the state has adopted the policy of leaving the construction and management of the railways to private companies, it has, nevertheless, intervened, by means of subvention or other encouragement, to stimulate private enterprise in those cases in which the lines run through localities where the commerce is deemed insufficient to produce the average profit on the capital invested. In different countries this object is accomplished by different expedients.

In some, a subvention in money is directly given; in others, the state takes a certain proportion of the shares, supplying the corresponding amount of capital on favorable terms; in others, the state guarantees a minimum amount of interest on the capital to be invested; in others, the companies are favored by the free importation of stock and materials by the gratuitous use of the land, and by exemption from taxation.

The authority of the state is, in almost all cases, asserted, and in many periodically exercised. Thus a power of revising the tariff at stated intervals, such as every three or five years, is often reserved. This is the case in some of the railway enterprises in the United States.

The case of the English railway companies is, in several respects, peculiar. The spirit of the laws and traditions renders the state averse from interference in commercial enterprises, and somewhat reserved even in the exercise of that control over them which would seem to be indispensable to the general interest.

Powers of an unusually extensive and durable character were, therefore, readily granted to all railway companies in this country, and monopoly after monopoly grew up, fostered by the legislature, and favored by the public. Monopoly, however, was not slow to develop some of its customary evils, and complaints and remonstrances followed. Abuses were signalized, and a reaction in public opinion was manifested. Railway directors, who had been previously the objects of unbounded laudation, now became the subjects of distrust and censure, and a general demand of some efficient system of control was put forth.

This demand was opposed by railway directors and parties under their influence, who went so far as to deny the right of Parliament to interfere with their concerns, assimilating their establishments to those of banks, insurance offices, dock companies, and other industrial associations. These parties indignantly rejected all control, and even complained of the system of publishing periodical reports, partial and imperfect as it has been, which the law and public opinion has exacted from them, as a grievance. They declared that any interference with the affairs of railway companies, or any compulsory pub-

lication of their proceedings, or any report of the state of their financial concerns, is a violation of the rights of capital as gross and unjustifiable as would be the same measures if adopted in reference to the mercantile transactions of Rothschilds, Barings, or any other private establishment. They admit that government may so far interfere as to provide for the safety and convenience of the public in travelling. But beyond this, they denounce all legislative or state intervention in their affairs. They complain that the temper evinced by Parliament and the press is such as ought to be directed only against the greatest enemies of social progress, instead of the promoters, as they justly enough claim to be, of one of the most signal instruments for the advancement of civilization that modern times have witnessed. Such a temper, they contend, must produce a corresponding feeling on the part of railway directors; and it is declared that, if such a system of annoyance and improper interference be continued, it must result either in raising a spirit of opposition on the part of railway interests, which, considering the magnitude of the property at stake, cannot be lightly regarded, or inducing an apathy and indifference in the administration of railways; in either case being the cause of great injury and inconvenience to the public.

To all this it is answered, that bodies which possess the almost exclusive control of the intercourse of the country, including the conveyance of persons and goods, the service of the post-office, and the movement of the troops have none of the qualities, and ought to have none of the privileges, attaching to private commercial establishments; that, therefore, it would be a great error to regard the British railways as speculations important to none but the shareholders; that they, on the contrary, involve interests public, political, and social, of the greatest magnitude; that they have not been created, as the advocates of their complete independence pretend, by the unaided efforts of individuals; that they owe their origin and existence to the will of the legislature, expressed in their various acts of incorporation, and that to the legislature they must be held, in a peculiar degree, responsible; that they have been intrusted with privileges and powers almost without precedent; and that, in fine, it is incumbent on Parliament to see that these powers are properly exercised, and to amend the laws which regulate them in such a manner as may from time to time be deemed expedient.

It is further contended, that the duty of legislative interference is rendered more imperative by the enormous amount of money which railway companies have raised under parliamentary authority. Not only has a capital been raised amounting to a quarter of the national debt, which amount will be augmented by at least fifty per cent. within a short period, but loans have been obtained by the companies to vast amounts, under the direct sanction, and subject to the conditions, of special acts of Parliament. The debentures representing these loans, as well as the railway shares, are transferable from

hand to hand with as much facility as the unfunded debt, with which they enter into direct competition.

Of late years, moreover, the interests involved in railway property have assumed an importance which has indroduced it into marriage settlements, wills, and other family arrangements, almost as generally as the public securities. It would, therefore, it is contended, be preposterous to maintain that property of such an amount and such character should be left to the uncontrolled management of bodies so fleeting and so little responsible as the boards of railway directors.

It is further maintained by the advocates of government control that shareholders are a fleeting and mutable body, liberated from many of the responsibilities and obligations which attach to property of a more permanent character. A share market has been created as well in the chief commercial towns as in the capital, where transactions to an enormous amount take place. Not only are permanent investments made in railway securities which have become matters of settlement, bequest, and inheritance, but large speculations are daily made, with a view to profit, by traffic in a description of property peculiarly liable to sudden and extraordinary fluctuations, — fluctuations so extreme that the capital of a single railway has been known to fall in value within the brief period of two months to the amount of three millions sterling. These violent and sudden variations in the value of the securities of one railway produce sympathetic effects in all the others, and always arise from the want of confidence entertained by the public in the representations made by the directors of railway companies of their financial condition.

Since, however, the necessity of establishing an independent body, invested with definite powers to examine and check the railway accounts, is admitted by all persons beyond the immediate circle of railway directors, and those in their employment and under their influence, and even by some among those directors themselves, it will not be necessary to enter further into this discussion. It may be assumed that the establishment of such a controlling body is demanded by public opinion ; the only points to be considered being the authority from which its nomination must emanate, and the nature and extent of its powers.

The appointment of such a body can only be made by the directors, by proprietors not directors, or by the state.

That railway directors should nominate the body which is to control themselves, would be an outrage on common sense, which public opinion indignantly rejects.

The appointment of an efficient and independent board of control by railway proprietors, exercising its powers over railway directors, would be attended with many practical difficulties. The railway proprietors are a very numerous body, scattered over the country, and even over the world, varying extremely

in age, sex, and condition. It is difficult to imagine how such a body could ever be brought into any real co-operation otherwise than by the agency and influence, direct or indirect, of the directors themselves. The body of proprietors have already nominated the directors, and must be presumed to have selected the individuals for that office whom they regarded as best entitled to their confidence. To call on the same proprietors to elect other individuals to be placed in a sort of antagonism to the former, and invested with powers to check and control them, would be to require them to place over those individuals, in whom they have manifested the greatest confidence, others in whom they must necessarily have less.

The impracticability of attaining such an object is in some degree illustrated by the effect of the system of audit hitherto pursued. It is well known that on the presentation of each half year's report, auditors are appointed by the meeting of shareholders to examine and to check the balance-sheet. The witnesses produced before the House of Lords, consisting of public accountants, eminent railway directors, and others distinguished by special knowledge on such subjects, were unanimous in declaring this system of audit to be destitute of all efficiency.

A board of railway control, properly constituted, would represent, not the interest of the shareholders only, but that of the public; and among the abuses which it would become its duty to check, would be more especially those which affect that portion of the public who are not shareholders. The misapplication of capital and financial malversations which have been already sometimes practised by directors, having the effect of producing factitious changes in the marketable value of railway securities, of which changes the directors themselves, who thus brought them about, have largely availed themselves, are examples of this. So far, then, as such a board of control would represent the interests of the public in general, as contradistinguished from those of railway proprietors in particular, it ought legitimately to derive its appointments and authority from the state, which represents the public.

But whatever may be the origin of such a controlling or auditing body, it is agreed on all hands that it must be perfectly independent of the directors in the exercise of its functions. If such independence can be shown to be compatible with any system of election by shareholders, no legitimate objection can, perhaps, be brought against it; and it would, in such case, be exempted from those inconveniences which are supposed to attend such a body when deriving its nomination and authority from the government.

Whatever may be the nature of the functions and limits of the powers to be conferred upon the body proposed to be created for the control or audit of railway management, its objects may be briefly and clearly stated.

They must be to supply railway shareholders, and the public in general (any of whom may at any moment become railway shareholders), with the means

of obtaining an assurance of the honesty and of estimating the ability of the railway management. This object will be attained partly by the confidence which the public may entertain in the persons appointed to compose such a board, and partly by the publicity which may be given to the accounts and proceedings of the railway managers.

One of the objects most strongly insisted upon in the measure proposed in the House of Lords for an independent system of railway audit in the session of 1849, was to secure a greater uniformity and more detailed explanation in the system of financial accounts issued by the directors to the shareholders. These accounts naturally arrange themselves under the two heads of capital and revenue.

It was proposed that the capital account should be twofold, or, to state it more correctly, a single account consisting of triple columns.

The first column would consist of a clear and detailed statement of the amounts of capital which the company had been authorized to raise, stating the purposes to which these amounts respectively had been directed by the legislature to be applied.

The second column would contain a statement of the extent to which the company had exercised these powers. It would state the amounts respectively which had been raised under each authorization, assigning them to their respective heads, and showing the purposes for which they were destined.

The difference between the totals of these two columns would show the amount of the unexhausted power with which the company was still invested.

The third column would contain a clear and detailed statement of the capital which had been actually expended, stating the objects to which it had been appropriated, and showing clearly that these objects were those for which Parliament had authorized the capital to be raised.

The difference between the totals of the second and third columns would show the portion of the capital raised which had been still unexpended.

One of the abuses against which legislative interference had been invoked, was the misappropriation of capital by railway directors. This misappropriation was twofold. In some cases the directors would apply the capital which the company had been authorized to raise for one purpose to another, still, however, being legitimately capital. Thus, capital authorized to be raised for the construction of a particular branch of the trunk line, would be applied to the purchase of steamboats, or to the improvement or construction of docks.

Such proceedings involved a double violation of the spirit of the law. Not only was capital applied to a purpose not authorized by Parliament, but works, the construction of which was sanctioned by Parliament, and ordered to be executed within a given limit of time, were left either incomplete or not commenced.

But the most frequent and scandalous misapplication of capital, whether considered in itself or its consequences, had been the appropriation of capital to the purposes of revenue, and more especially to the payment of dividends.

Railway directors are usually large holders of shares, frequently obtained by allotment, and at a much lower rate than the current market price.

Thus situated, they have a direct interest to raise the market, and to avail themselves of such elevation to dispose of shares.

This object is accomplished by the misappropriation of capital for the purpose of swelling the dividends beyond the amount which they would have reached if paid legitimately out of profits.

When a rise has been produced by these means, and the directors avail themselves of it, they dispose of their allotted shares at a large profit. This spurious price is of course only temporary, and the market soon declines. The deluded public loses precisely to the extent to which the directors and those in their confidence gain. Thus the fortunes of the widow and orphan, and the accumulations of industry and thrift, are fraudulently transferred to swell the colossal fortunes of individual directors, who by such means suddenly rise from stations comparatively obscure to almost fabulous wealth.

It may be most truly replied, that proceedings such as these are rare, that directors in general are persons altogether incapable of such malpractices, and that it would be unjust to stigmatize a large, respectable, and intelligent body of men, to the unwearied exertions and talents of many of whom the world is indebted for the successful issue of the most signal improvement of modern times, because of the misconduct of some individuals among them. To this it is answered, that unreserved and complete publicity of all the details of the management of the affairs of each company can alone do justice to the respectable and independent majority of directors. Such a publicity will enable every one who possesses the necessary information to judge not only of the honesty but of the ability of the management, and without such publicity there can be no test by which the public at large can know the integrity or skill with which any railway establishment is conducted.

An intelligent and experienced witness, long connected with railway affairs. declared, before the committee of the House of Lords, that practices of misapplying capital, such as had prevailed in certain cases would lead at some period to "total ruin, and in the meanwhile to great confusion, and an entire misapprehension of the value of each undertaking."

Another said, that there was "no safety for bondholders or shareholders, unless the separation of capital from revenue was observed, and that any deviation from it must falsify the accounts, and deprive the public of the means ot measuring the value of such undertakings."

An experienced accountant stated, that under the present system "there is no security that capital and income shall be kept distinct, and that the practi-

cal consequence is, that the purchaser who buys shares does so in ignorance of the true state of the company's affairs, and is led to give a higher price than the thing is worth, under the belief that the dividends declared come *bonâ fide* out of profits. Any balance, under such a system, may be struck which may suit the purpose of the directors; any dividend may be declared, and the public may be deceived to any extent desired."

"If capital," says the Report of the Lords' Committee, "be unduly brought to increase income, or ordinary expenditure be unduly carried to the account of capital, the apparent balances may be varied at pleasure, a fallacious and fraudulent value may for a time be given to shares, greatly profitable to all proprietors desirous of selling, but leading to results fatal to the interests of the more important class who invest permanently; for the sake of a deceptive present gain the value of the reversion will be sacrificed. Cases may easily be contemplated, and undoubtedly have occurred, in which the future profitable working of the line may thus be endangered, and the public interests connected with the maintenance of railways be placed in jeopardy, if not sacrificed."

To guard against this and similar abuses, shareholders have always had a certain power at reasonable times to examine the books of the company, but this power has proved, as might easily have been foreseen, illusory. It is not by individual shareholders going to a railway office, and demanding journals and ledgers, and running over their pages, that any real estimate of the state of the affairs of the company can be ascertained. This is a proceeding which individual shareholders will never be induced to undertake, nor, if they did, would any satisfactory result ensue. Practised accountants alone can form a satisfactory estimate of the financial condition of the company, and even they could only accomplish this by an elaborate examination of the books; such an examination as individual shareholders could never effect by the means provided in the acts of incorporation.

But whatever powers may be conferred upon the controlling or auditing body, and from whatever source it may derive its appointment and authority, its influence will be unavailing unless the most ample and unreserved publicity be given to the details of the railway management, and with such publicity the task of the auditors or controllers will be rendered comparatively easy. Their duties will, in such case, be reduced in effect to mere verification of the disbursements by the vouchers; for, by such means, the public at large would be converted into one great and unquestionable board of audit. Railway affairs would, in a word, be placed under the immediate operation of public opinion. Railway directors, instead of demanding, as they now do, half-yearly votes of confidence from their blindfolded constituents, would receive the intelligent approbation of a well-informed public.

In all the discussions which have hitherto taken place on this question of railway control, a stress much too exclusive has been placed on the fidelity

and accuracy of the report of the financial condition of the company, as if the honesty and integrity of the management were all that could be required to satisfy the railway proprietors and the public. The degree of ability and skill with which the affairs of the railway may have been conducted, seems to be wholly left out of view. This is a grave error. Honesty is happily a much more ordinary quality than ability, and there is much stronger ground for distrusting the skill shown in the management of the enterprise of a railway than the integrity of those to whom the management is confided.

It is not, therefore, sufficient, for the satisfaction of public opinion, to publish an authenticated report of the financial conditions of each railway company.

Such details of its management must also be given as may enable all persons competently informed to form an estimate of the skill and ability with which its affairs have been conducted. They must be in a condition to judge whether the capital has been duly utilized; but, to place them in this condition, a much more ample report of the business of the company must be published than any which has hitherto been issued by railway companies in England, or even on the continent, where the periodical reports are more detailed. The Belgian government alone puts forth a complete and satisfactory annual report of its management. We do not maintain that the exposition annually supplied to the public by the Belgian government of the administration of the state railways may not be susceptible of improvement, or that it may not contain some needless detail. It cannot, however, be denied that it demonstrates the possibility of placing the affairs of railway management under the operation of public opinion.

For returning the details of railway operation in such a manner as to enable useful conclusions to be drawn from them, the following outline for an annual report is proposed: —

Section I. — Construction and Stock.

Sums which the company has been empowered to raise. Sums actually raised under such powers. Sums expended, specifying in detail the objects to which they have been appropriated, and the sources from which they have been derived.

Section II. — Expenses.

This section should contain a detailed statement of the current expenses of the management and working of the railways, each class of disbursement being assigned to its proper head — such as direction and management, way and works, locomotive power, carrying expenses, etc.

Section III. — Receipts.

This section should contain a detailed statement of the revenue of the company, assigning distinctly the amount of revenue proceeding from each object of traffic, such as passengers, distinguished by classes, baggage, parcels, horses, carriages, mails, and all objects carried by passenger trains; goods and live stock, classed according to their tariff.

The receipts should also be stated according to the parts of the line from which they have proceeded; thus the amount received for each class of traffic at each station should be given.

The receipts should also be classified according to the period of the year at which they have been realized, their amounts being separately stated for each successive month.

In cases where a graduated tariff has been established, diminishing as the distance to which the objects of transport are carried is increased, the receipts should also be classified according to the distances to which the objects of transport producing them have been severally carried, so as to show the amounts of revenue which have proceeded from long traffic and short traffic.

Such a statement is supplied in the reports of the Belgian railways.

Section IV. — The Movement of the Traffic.

This section should contain a statement of the quantity and mileage of the several classes of traffic. Thus, the number of passengers of each class booked, and the total mileage of each class should be given. In like manner, the quantity and mileage of each object of transport conveyed by passenger trains, such as baggage, parcels, mails, horses, and carriages, should be stated. The comparison of the quantity of these with their mileage would give the average distance over which each passenger and other object of traffic was carried.

A like statement should be given for the various classes of goods traffic, showing in each case the quantity booked, and its mileage.

The quantity booked at each section of the line should be distinctly given, to show the variation of the traffic on different parts of the railway; and the quantity in each month, to show the variation of the traffic according to the seasons.

Section V. — The Movement of the Locomotive Stock.

This section should contain a statement of the quantity of the locomotive stock, enumerating the engines, with the circumstances of their origin, con-

struction, age, former services, and their current mileage. The distances run by each engine during the year should be stated, as well as the total distance it has run since first put upon the road. The consumption of fuel should be given, distinguishing that which is consumed in lighting and getting up steam, and standing, and in profitable work. The consumption of oil and other materials, and the cost of repairs, should also be given. All these details are supplied annually in the reports of the Belgian railways.

Section VI. — The Movement of the Carrying Stock.

This section should contain a statement of the entire stock of vehicles of transport used during the year, distinguishing them according to classes, and giving their mileages respectively.

Also a statement of the consumption of materials, cost of repairs, etc.

Section VII. — Movement of Traffic compared with Movement of Locomotive and Carrying Stock.

By comparing the movement of the different classes of traffic with the movement of the various classes of vehicles of transport to which they are respectively appropriated, we can obtain the average load carried by each vehicle, and by comparing them with the movement of the locomotive stock, we can obtain the average load drawn by each engine. Data are thus obtained by which numerous economical problems of the highest importance can be solved. It is by these means that we can ascertain the extent to which the moving stock of the railway has been utilized.

Section VIII. — Receipts and Expenses compared with the Movement of the Traffic and Rolling Stock.

By this comparison may be ascertained the proportion of the expenses chargeable to each class, and even to each individual object of traffic. By comparing such expenses with the receipts arising from each object of traffic, the profit or loss arising from each class of traffic can be ascertained.

By this means a numerous class of important problems can be solved which are intimately connected with the questions of the tariff, and by which alone the future tariff can be advantageously regulated.

Section IX. — The Movement of the Traffic and Rolling Stock compared with the Extent of the Railway.

The comparison made in this section would show the extent to which the railway itself has been utilized. It would indicate the proportion in which the traffic has been distributed over it, showing the quantity of profitable load as well as of dead weight which has been transported between station and station on every part of the line. This would also indicate the extent to which the local supply of traffic may have been cultivated, and would direct the attention of managers and the public to the still unsatisfied exigencies of the districts through which the railways may be carried.

It must not be supposed that a report containing the details enumerated here, is either difficult or impracticable.

Many of them are regularly supplied in the annual reports of most of the continental railway companies, and all of them, and many others still more minute, are contained in the annual railway report of the Belgian government. It is true, that the existing arrangements of the English railways do not afford the means of recording some of these statistical facts, but nothing would be more easy than to organize in this country, as elsewhere, the means of recording them.

In order to show the extent to which the movable stock of the railway has been utilized, it is essential to supply the means of comparing the movement of the rolling stock with the movement of the traffic. It is by such a comparison alone that the average amount of loads carried by the different vehicles of transport can be accurately ascertained.

If we know the distance travelled by any special class of vehicles of transport within the year, and also know the distance over which each class of objects of transport to which such vehicles are appropriated has been carried, the comparison will immediately supply the means of ascertaining the average load carried by each vehicle, and this average load is the only exponent of the extent to which each class of vehicle has been utilized.

To accomplish this it would be necessary to keep separate mileage accounts of the traffic and of the rolling stock. In the case of the traffic, its mileage can be immediately ascertained from the record of the receipts, inasmuch as each sum received represents the transport of a given object to a given distance.

In the case of the vehicles of transport, the manner in which the mileage has been hitherto kept on continental lines is not as simple and satisfactory as could be desired. The places of departure and arrival of each vehicle are registered, and reports from the different stations are received, the comparison of which supplies the means of computing the mileage.

Nothing, however, would be more easy than to attach to each vehicle of transport a *counter*, which would become a self-acting register of the aggregate space over which each vehicle has run. These counters, when required in large numbers, could be constructed at a small expense. They are not liable to derangement, and would relieve the railway administration from the clumsy and expensive method of observing and registering the movement of the stock, and, in fine, would accomplish the object with greater certainty and accuracy. The counters, as commonly constructed, run up to a million of revolutions of the wheels, which, with a wheel ten feet in circumference, would, in round numbers, extend to about two thousand miles.

Similar instruments might be attached to the engines, by which a register of their mileage would be kept. In this manner an account recorded of the movement of the entire rolling stock would be obtained at a nominal expense, nothing more being necessary than to provide agents who would attend to and record the indications of the counters.

The expenses, besides being recorded under the usual heads of direction, way and works, locomotive power, carrying expenses, etc., should also be distributed so as to enable the managers of the road to ascertain the cost at which each object of traffic has been transported. It is by a comparison of this cost with the tariff, that the profit arising from each object of traffic is ascertained. Data would thus be also obtained, by which the managers could ascertain what increased expense would be produced by any given increase of the distance to which each object of traffic is transported, and hence would arise the data necessary for the formation of a graduated tariff, diminishing in its rate per mile according as the distances to which the objects of traffic respectively are transported are increased.

These, and a multitude of other practical problems, involving the most important economical principles in railway management will at once suggest themselves as arising out of the circumstances here adverted to, and the solution of which would be altogether impossible unless data, such as those here described, could be obtained.

No such data can be obtained, however, from the present system of railway accounts, nor is it possible for directors and managers themselves to obtain the means of solving such economical problems.

Connected with each railway administration, a statistical bureau should be established for organizing and recording these classes of data.

Such bureaus are already established in connection with several of the best conducted continental railways, and although their operations have not been in all cases conducted so efficiently as could be desired, they are, nevertheless, attended with the best effects.

It is sometimes contended that railways, being commercial companies, whose concerns affect only their respective shareholders, publicity should not be exacted from them, and that the shareholders alone have a right to be

informed of the affairs of their administration and management; but to this it may be answered, that nothing short of publicity can bring such information to the knowledge of bodies so large and fluctuating as those of railway shareholders.

Besides, it may be answered, the shares being matters of daily bargain and sale in the public market, every individual who may become a purchaser has a claim to a full knowledge of the state of the affairs of the company into which he is about to enter.

In fine, considering the questions which have been agitated for some months respecting the great railway enterprises of the country, in all their bearings and relations, no expedient appears so likely to remedy the evils which have formed the subject of universal complaint and remonstrance, to revive public confidence, and to restore railway property to its just value in the market, as a system of publicity, such as here recommended.

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